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This proceedings includes 22 papers on topics related to the development and application of computer-based mathematical models for water quality and quantity management. The papers were presented at the semi-annual meeting of the Joint U.S.-Canadian Stormwater and Water Quality Model Users Group held on March 24-25, 1986, in Orlando, Florida.

Several papers deal with using stormwater and water quality models on microcomputers and interfacing microcomputer software such as spread sheets and data base manageers with these models. Specific programs discussed include the Storm Water Management Model, DABRO, HSPF, a simplified water quality program, HAZPRED, QUAL-TX, and OTTSWMM.

Other papers discuss statistical properties of point and nonpoint pollutant sources, particularly highway runoff and the effectiveness of detention/retention basins for mitigating pollution. Two papers discuss trophic state models in lakes and reservoirs.

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OF
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USERS GROUP MEETING

March 24-25, 1986
Orlando, FL

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FOREWORD

A major function of research and development programs is to effectively and expeditiously transfer technology developed by those programs to the user community. A corollary function is to provide for the continuing exchange of information and ideas between researchers and users, and among the users themselves. The Stormwater and Water Quality Model Users Group, sponsored jointly by the U.S. Environmental Protection Agency and Environment Canada/Ontario Ministry of the environment, was established to provide such a forum. The group has recently widened its interests to include models other than the Stormwater Management Model and other aspects of modeling water quality in urban and natural waters. This report, a compendium of papers presented at the users group meeting held on March 24-25, 1986 in Orlando, FL, is published in the interest of disseminating to a wide audience the work of group members.

Rosemarie C. Russo, Ph.D.
Director
Environmental Research Laboratory
Athens, Georgia

ABSTRACT

This proceedings includes 22 papers on topics related to the development and application of computer-based mathematical models for water quantity and quality management. The papers were presented at the semi-annual meeting of the Joint U.S.-Canadian Stormwater and Water Quality Model Users Group held on March 24-25, 1986, in Orlando, Florida.

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ACKNOWLEDGMENT

The Stormwater and Water Quality Model Users Group relies on local hosts to make arrangements for its meetings. The hosts for the meeting reported in this proceedings were Dr. Larry A. Roesner of Camp, Dresser, McKee, Inc. and Dr. Wayne C. Huber of the University of Florida. Dr. Huber reviewed abstracts of papers and arranged the meeting agenda. Dr. Roesner made local arrangements for meeting rooms and hotel accommodations.

"Porting Mainframe-Based Numerical Models to Microcomputers: A Case Study Using the EPA Storm Water Management Model"

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Metcalf & Eddy, Inc., Wakefield, Massachusetts

ABSTRACT

This paper describes porting of the United States Environmental Protection Agency Storm Water Management Model (SWMM Version 3) from an IBM mainframe CMS environment to a DOS environment on an IBM PC AT microcomputer. Subsequent application of the micro SWMM model during a study of a combined sewer system in a major New England city is then briefly reviewed.

INTRODUCTION

As a result of the rapid evolution of micro hardware and recent advances in the development of micro-resident Fortran compilers, porting and application of traditionally mainframe-based numerical models on micros has become a practical and extremely cost effective method of computing. The advantages of computing on micros are numerous and not restricted to cost considerations alone. They include: easier, more user friendly access, possible reduced turn-around times and integration of pre-processing, numerical modeling, post-processing, word processing, graphics and software development capabilities within one, relatively compact machine.

Prior to porting of the SWMM model to micros, engineers at Metcalf & Eddy executed it on a remote IBM 3033 commercial time-share computer. Input data sets were developed on an in-house Digital Equipment PDP 11/70 and transmitted to the remote IBM mainframe via telephone line. Following execution of SWMM on the mainframe, model results were stored on disk and subsequently transmitted back to the in-house PDP 11/70, where printing was accomplished using a line printer.

Use of this remote job submittal system was satisfactory as long as only one or two users were active at once. However, during periods of heavy use, turnaround times increased drastically from the usual 30 minutes to as much as three hours. In addition, if either the in-house PDP 11/70, remote mainframe or telephone communications were down, computing would come to a halt.

It was recognized that several options were available for decreasing turn-around times on the above remote job submittal system. However, these options would not address cost and dependability issues. Due to several previous successes in porting relatively small numerical models to micros at Metcalf &

Eddy, porting of the SWMM model to in-house micros was initiated in an attempt to address the issues of cost, dependability and turn-around.

THE PORTING PROCESS

The process of porting numerical models from mainframe (or mini) computers to micros can be divided into several major tasks, including:

Obtaining the mainframe source code,
Transferring source code to the target micro,
Selecting a micro-resident compiler,
Modifying and compiling the source code,
Selecting a micro-resident linkage editor,
Developing an overlay structure and linking object modules,
Testing and documenting the ported numerical model, and
Applying the model on projects.

The above tasks are more or less the same regardless of the relative size or complexity of the mainframe model. In addition, the porting process is independent of the high level language in which the model source code is written, e.g., Fortran, Pascal or C. However, selection of a micro-resident compiler and linkage editor and development of an overlay structure are best described using a large, relatively complex numerical model like SWMM. As a result, the following detailed description of the porting process for SWMM should serve as a useful guide to others interested in utilizing the rapidly expanding capabilities of micros.

OBTAINING MAINFRAME SOURCE CODE

Mainframe source code for the latest version of the SWMM model was obtained on 9-track magnetic tape from the USEPA Environmental Research Laboratory in Athens, Georgia. This tape included the main SWMM program and all subroutines and functions. It also contained test input data sets and results for use in verifying model operation. A listing of all files on the SWMM tape was produced on an in-house line printer. The SWMM source code was examined and found to be complete.

Each subroutine contains its own COMMON BLOCK statements. However, it is important to note that many models incorporate COMMON BLOCK specifications and other source code segments, e.g., PARAMETER statements, into the main body of code at compile time through the use of INCLUDE statements. Thus, it is necessary to obtain source code for all of these included files too.

Model documentation and users manuals for SWMM were obtained from Dr. Wayne C. Huber at the University of Florida in Gainesville. From an examination of these manuals, it was found that the mainframe SWMM model utilizes overlays at run time to reduce the size of the region in which the model executes to approximately 400 kilobytes (kb) of random access memory (RAM). As will be shown later, use of the above overlay structure for the micro SWMM model eliminated the need to develop one from scratch.

A description of the job control language (JCL) used with

SWMM on the IBM mainframe OS/VMS (Release 3.8) operating system was also included in the documentation. This information was useful in identifying input and output (I/O), e.g., scratch files, used during SWMM model execution.

Documentation for the Fortran compiler used with the mainframe SWMM model was also obtained. This information was used later in the porting process during selection of an appropriate micro-resident compiler. In general, if the mainframe Fortran compiler used with the model to be ported uses features not included in the Standard Fortran-77 Language (ANSI X3.9-1978), then the mainframe compiler documentation should be obtained as an aid in porting to micros.

TRANSFERRING SOURCE CODE TO THE TARGET MICRO

The 9-track tape containing the SWMM source code and test data was mounted on an in-house PDP 11/70 tape drive and copied to a user area on a peripheral disk. The numerous SWMM files were then merged using the PDP 11/70 operating system APPEND utility.

Transfer of the master SWMM file from the PDP 11/70 to the target in-house IBM PC AT hard disk was accomplished using Hayes Smartcom communications software and a micro-based internal smart modem. Due to the use of relatively slow (1200 baud-rate) modems at both the PDP 11/70 and target micro, transfer of the master SWMM file required almost four hours.

Transfer of mainframe source code to target micros may be accomplished using several other methods, including: direct modem-based communication between mainframes, minis and micros, magnetic tape and a micro-based tape subsystem or micros hard-wired to minis or mainframes.

Service bureaus may also be used to copy data from magnetic tape to target micro-compatable floppy diskettes. As a last resort, hard copy of the mainframe source code can be entered manually into the target micro using a micro- resident text editor. When micro-based tape subsystems become cheaper and more versatile, their use for transferring mainframe data to micros and for backing up micro-based hard disks and floppy diskettes will likely become commonplace. It is important to realize that whenever data is transferred between mainframes, minis and micros using magnetic tape, the compatability of tape read/write utilities on these systems must be addressed.

SELECTION OF A MICRO-RESIDENT COMPILER

Documentation for the mainframe Fortran compiler used with SWMM was examined. As expected, it was found to be a superset of the ANSI Fortran-77 standard. Information on the various language extensions available with this compiler was used later during evaluation of alternative micro-resident compilers.

The Microsoft Fortran compiler (Version 3.2) had been previosly applied successfully at Metcalf & Eddy during porting of several small numerical models from Digital Equipment minicomputers to Intel 8086 processor based 16-bit micros. As a result, compilation of SWMM was initially attempted using this compiler. Unfortunately, Microsoft Fortran was unable to compile several of the larger SWMM subroutines, e.g, RHYDRO and TRANS, due to its 64 kb limit on program size. In addition, this compiler's memory model forced all local data, i.e, variables used locally within subroutines, in the entire SWMM model to reside within one 64 kb local group (DGROUP) in micro RAM. The 64 kb limitation on program size was overcome by splitting up the several very large SWMM subroutines. However, the above DGROUP restriction resulted in DOS "stack overflow" and " heap" errors at model run time.

Due to the above limitations of the Microsoft Fortran compiler, a search was initiated for another compiler. This search resulted in selection of the Ryan-McFarland Fortran (RM/FORTRAN 2.0) compiler for use in porting SWMM. RM/FORTRAN, which is also marketed by IBM as Professional Fortran, is available for either DOS or XENIX operating systems. It uses a memory model which overcomes the 64 kb DGROUP restrictions of Microsoft Fortran and allows program sizes up to sixteen megabytes. RM/FORTRAN is certified by the General Services Administration (GSA) as the only micro-resident Fortran compiler that meets the full ANSI X3.9-1978 Fortran Standard without discrepancies. RM/FORTRAN includes many frequently used mainframe Fortran language extensions, including those used in Digital Equipment VAX/Fortran, IBM mainframe VS/Fortran and Fortran-66. RM/FORTRAN applications run 30% to 40% faster than those compiled with Microsoft Fortran Version 3.2. A numerical coprocessor chip (Intel 8087 for IBM PC or XT and Intel 80287 for IBM PC AT) is required when using RM/FORTRAN or when running RM/FORTRAN compiled numerical models.

MODIFYING AND COMPILING SOURCE CODE

The master SWMM file on micro hard disk was separated into its component files, i.e., data files, main program and subroutines, using a BASIC language program developed on the target micro. This program was also used to convert several mainframe language extensions used in SWMM, which are not available in RM/FORTRAN, and to separate out all COMMON BLOCK statements and related PARAMETER statements into INCLUDE files.

Use of include files often results in considerable time savings during model testing, debugging and modification because COMMON BLOCK array dimensions and variable lists modified in one INCLUDE file result in global changes to the model. However, when splitting out COMMON BLOCK statements into INCLUDE files one must be cognizant of the fact that some programmers rename variables within subroutine COMMON BLOCK statements instead of formally equivalencing these variables. Several instances of this practice were found in SWMM. These locally renamed COMMON BLOCK variables were respecified using appropriate EQUIVALENCE statements.

An examination of documentation for the mainframe SWMM model indicated that control of external data files, e.g., interface and input data files, is maintained through the use of IBM Job Control Language (JCL). In contrast, file control using the DOS implementation of RM/FORTRAN is accomplished by Standard Fortran OPEN and CLOSE statements within the model source code. Accordingly, the SWMM source code was modified to include statements necessary for creation and control of all external data files. Specifications for these files, including: logical unit numbers, access modes (sequential versus direct), format (formatted versus unformatted) and status (named versus scratch), were determined from a detailed examination of their corresponding source code READ and WRITE statements. The example JCL specifications included in the mainframe SWMM documentation and user manuals were also useful for establishing the characteristics of these files.

All SWMM subroutines were then successfully compiled using RM/Fortran. This compilation, which was automated through the use of a DOS batch file, required approximately two hours to complete. The total size of the resultant object modules was over a megabyte.

SELECTING A MICRO-RESIDENT LINKAGE EDITOR

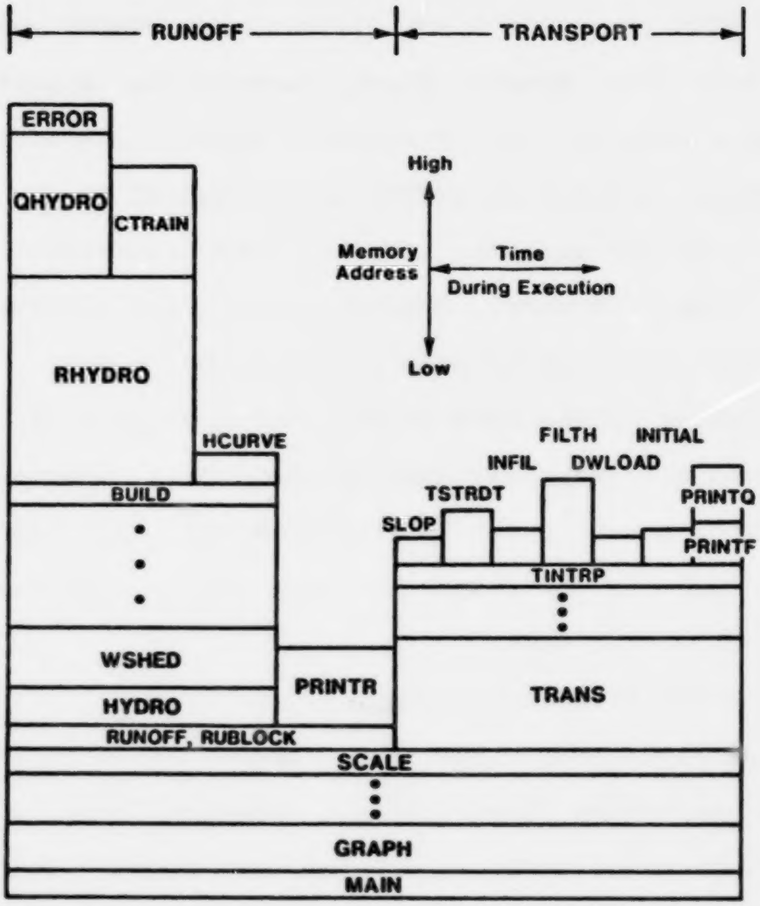
Selection of an 80286 processor-based linkage editor to be used on the target IBM PC AT involved several considerations. The requirement of compiler-linker compatability did not present a problem, since RM/FORTRAN may be used with numerous linkage editors. However, during the initial attempt at linking the Microsoft compiled SWMM model, it was found that the Microsoft Linker supplied with DOS had two serious deficiencies.

First, it was found that the DOS linker can only link up to 900 kb of object modules into an executable program. Thus, if the DOS linker were to be used, SWMM would have to be separated into several blocks and each block would have to be executed as a separate model. This would likely prove to be very inconvenient during model application.

Second, it was found that the DOS linker can not be used to build multiply-nested overlay structures. As stated previously, four overlay nesting levels were required for execution of the mainframe SWMM model within a 400 kb region of RAM. Since DOS can only address 640 kb of micro RAM, the need for multiply-nested overlays with the micro SWMM model was seen as a distinct possibility. Accordingly, a search was initiated for a linkage editor which would overcome the overlay nesting and object module size restrictions of the DOS linker.

Based on the linkage editor needs identified above, the Phoenix Software Associates PLINK86 (Version 1.48) linker was selected for porting of the mainframe SWMM model. PLINK86 is compatable with RM/FORTRAN, supports up to 32 overlay nesting levels and does not have any practical restrictions on the total size of objects modules it can link.

The mainframe SWMM overlay structure was utilized as-is, following its conversion to PLINK86 format, to create the numerous overlays required for execution of SWMM within the 640 kb DOS addressable RAM partition. A memory diagram of a portion of the SWMM overlay structure is shown in Figure 1. The vertical scale represents the relative RAM memory addresses



RUNOFF
TRANSPORT
ERROR
QHYDRO
CTRAIN
RHYDRO
HCURVE
BUILD
WSHED
HYDRO
PRINTR
RUNOFF, RUBLOCK
SCALE
GRAPH
MAIN
High
Memory Address
Time
During Execution
Low
FILTH
INITIAL
INFIL
DWLOAD
TSTRDT
SLOP
PRINTQ
PRINTF
TINTRP
TRANS

FIGURE 1. MEMORY DIAGRAM SWMM V3 III

occupied by SWMM subroutine object code and data during model execution. In contrast, the horizontal scale indicates memory addresses (and subroutines) which are overlayed at various times during a model run. It is seen that at run time many of the memory addresses occupied by code and data of the SWMM Runoff Block are replaced, i.e., overlayed, by code and data of the SWMM Transport Block. This overlaying occurs at the instant when the SWMM Executive Block (subroutine MAIN) encounters a RETURN from the Runoff Block (subroutine RUNOFF) and then initiates a CALL to the Transport Block (subroutine TRANS). Also, while subroutine QHYDRO of the Runoff Block is executing in RAM, code and data for all of its ancestors, including: RHYDRO, HYDRO, RUNOFF, RUBLOCK, MAIN and all the RM/FORTRAN library routines used by SWMM must also be in RAM.

Linking of the SWMM object modules into a relocatable execute module required approximately five minutes. The SWMM execute module required 1.4 megabytes of hard disk storage. However, use of overlays resulted in a maximum 507 kb RAM requirement at run time.

TESTING OF THE MICRO SWMM MODEL

The micro SWMM model was then tested using the EPA supplied mainframe test cases. Results were found to be identical to those produced on the mainframe. Test case run times varied between approximately two minutes for execution of the Runoff Block to almost forty minutes for execution of the Extran Block. Results were printed at the rate of eight pages per minute on a Hewlett-Packard Laserjet+ Printer attached to the target IBM PC AT.

Mainframe models which utilize highly iterative solution techniques may exhibit significant computational error or even numerical instability when ported to micros. The reason for this is that round-off errors can accumulate to a higher degree when using 16-bit (typical micros) processors than when using full 32-bit processors (mainframes). This problem should be assessed during model testing. If deemed necessary, double precision may be specified for certain key variables used in iterative procedures of the micro numerical model. Test results for the micro SWMM model suggest that increased precision is not necessary.

When an overlay structure is developed from scratch for use with a ported micro model, it may be desirable to optimize that structure in an effort to speed up model execution. A general guideline to be followed during development and optimization of overlay structures is to set up overlays which contain isolated functional groups of code that will execute to completion and then will not be returned to for a long time, if ever. Also, the simplest overlay structure required to run within addressable RAM is the best. Stratcom Systems, Inc. presently markets a program which creates optimized overlay structures based on subroutine sizes and CALL sequence information produced by the RM/FORTRAN compiler. Use of this porting tool is currently being investigated at Metcalf & Eddy.

Following the successful testing of the micro SWMM model, the SWMM documentation was modified to provide users with instructions on its hardware/software requirements and its execution on an IBM PC XT or IBM PC AT. Since the micro SWMM model's functionality is identical to the mainframe implementation, no additional modifications to the EPA documentation and user manuals were required.

APPLICATION OF THE MICRO SWMM MODEL

The micro SWMM model was then used to simulate storm-induced flooding within a combined sewer system in a major New England city. Figure 2 shows the general configuration of the modeled sewer system, which was divided into 12 sub-catchment areas feeding into a sewer pipe system discretized into 12 sub-systems with a total of approximately 1500 computational elements. A schematic of the main sewer system SWMM model given in Figure 3 shows the hydraulic connections between the various sewer sub-systems .

Single event, 24-hour simulations were made using the SWMM Runoff and Transport Blocks and a modified version of the SWMM Combine Block capable of accepting up to 16 input hydrograph files. A ten minute time step was used in all

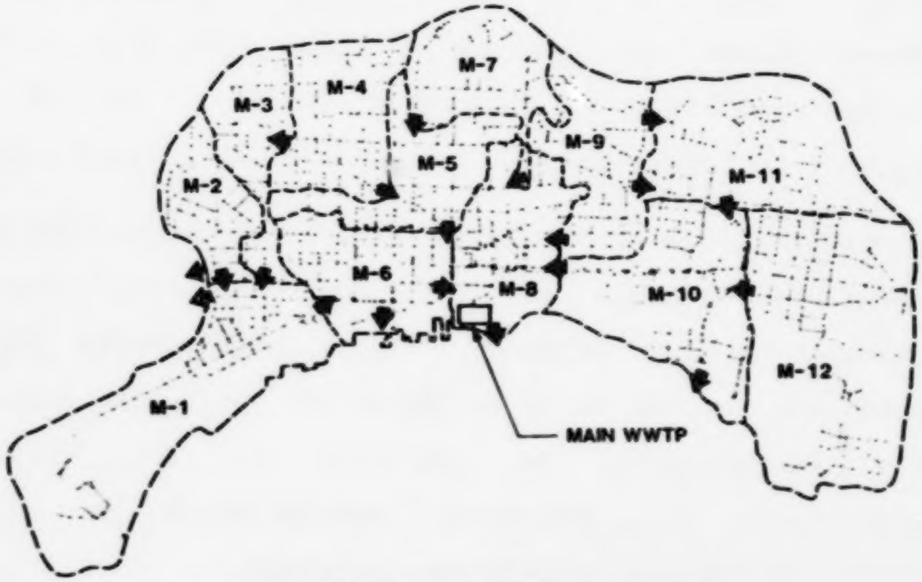
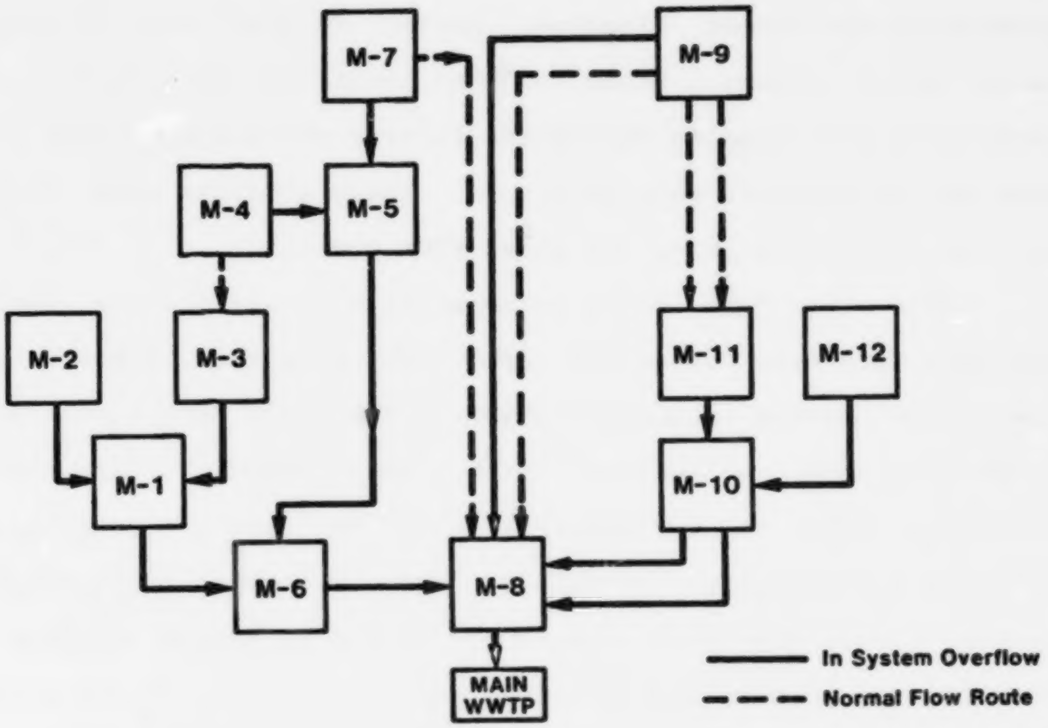


FIGURE 2. MAIN SEWER SYSTEM



M-7
M-9
M-4
M-5
M-2
M-3
M-11
M-12
M-1
M-10
M-6
M-8
MAIN WWTP
In System Overflow
Normal Flow Route

FIGURE 3. SWMM SUBAREA FLOW SCHEMATIC FOR THE MAIN SEWER SYSTEM

simulations and only flow quantity was modeled. Each of the twelve sub-systems of the total sewer system was simulated during individual SWMM runs. Batch files were constructed to control sub-system model execution and the cascading of outflow and in-system overflow hydrographs throughout the sewer network.

The SWMM sewer system model was then successfully calibrated and verified using two independent rainfall and in-system flow data sets and synoptic field observations of sewer system depths of flow and surcharging. The verified model was subsequently used to simulate in-system flooding, i.e., surcharging and street flooding, during 10-year and 50-year design storm events. Based on the locations and severity of surcharging and flooding during the 10-year storm event, several sewer relief projects were developed. These projects were then designed and tested using the micro SWMM model.

Both the SWMM model calibration and verification input data sets were also run on the remote IBM mainframe commercial time share system as a cross check of the micro SWMM model and to provide comparative run cost and speed benchmarks. Statistics for the verification runs on both the micro and mainframe are presented in Table 1. Model run time on the micro was much longer than that experienced on the mainframe computer. However, run cost on the micro was only 12% of that incurred on the mainframe.

TABLE 1: Benchmark Run Results

IBM PC AT		IBM 3033	
RUN TIME	RUN COST	RUN TIME	RUN COST
2.9 Hrs.	$20.	1.1 Min.	$175.

The longer run times experienced when using the micro SWMM model were not found to represent the critical path in the engineering analysis of the above sewer system. In addition, use of the in-house micro for SWMM modeling resulted in almost a 90% decrease in computing costs incurred during that project.

FUTURE DEVELOPMENTS

The porting and application of traditionally mainframe based numerical models on micros has some limitations. Very often model array sizes must be decreased in an effort to "fit" the model into addressable micro RAM. For example, the maximum number of finite element grid nodes allowed in a two or three dimensional hydrodynamic or water quality model may have to be decreased from 900 to 400. This may result in a micro model with rather limited applicability to the simulation of complex systems.

Another limitation of micro models is their relatively slow speed of execution. In most applications, e.g., simulations using any SWMM block except Extran, the slower execution times on micros do not represent the critical project path. However, in certain applications, use of micros may be prohibitively slow. Long-term, continuous-mode simulation of a large combined sewer system using the SWMM Extran Block is a good example.

Fortunately, there are several methods for increasing the potential model sizes and execution speeds on micros. The following discussion briefly reviews two of these methods as they relate to the micro SWMM model.

SWMM makes extensive use of external scratch data files, both within individual computational blocks and for communication between blocks. These scratch files, which may require over a megabyte of storage, are written to and read from high access speed drum disks on mainframe computers. These peripheral disks are also usually controlled by fast, dedicated processors. In contrast, these I/O activities are controlled by non-dedicated processors using relatively slow access floppy and Winchester hard disks on micros. As a result, use of hard disk scratch files by the micro SWMM model is very inefficient and no doubt contributed to the relatively long run times experienced on the micro.

Data transfer within micro RAM is much faster than data I/O using hard disks or floppies for storage. To take advantage of this fact, several virtual (RAM) disks could be configured on the target IBM PC AT using the DOS device driver VDISK.SYS. Resident RAM on the micro could then be increased from the current 640 kb to over three megabytes through the use of plug-in RAM chips mounted on its AST advantage board and an insertable piggy-back RAM expansion board.

Following the above reconfiguration of DOS and addition of RAM, file OPEN and CLOSE statements within the SWMM source code would have to be modified to include the new DOS configured virtual disk drive specifications for all scratch files. The SWMM executable module would then have to be rebuilt.

The above method could be used to speed up SWMM runs by eliminating much uneccesary, relatively slow I/O activity. It could potentially be used to increase the maximum size of sewer systems modelable on micros using SWMM by allowing the temporary storage of certain groups of variable arrays in virtual disk resident external data files located outside of the 640 kb DOS addressable RAM partition.

The same result could be more efficiently achieved through the use of a Unix or Unix-like operating system instead of DOS. Xenix operating systems are currently available for IBM PC's and many other 16-bit (and 32-bit) micros. Unix and Xenix systems do not have the RAM addressability restrictions of DOS. They are capable of addressing the entire amount of micro-resident RAM. Thus, all the SWMM scratch data files could be stored in RAM and the size of variable arrays, and hence model sizes, could be increased. It is important to note that the micro-resident Fortran compiler used during model compilation would have to be compatable with the target micro-resident Unix or Xenix operating system. Fortunately, a Xenix version of RM/FORTRAN has recently been released. Current efforts at Metcalf & Eddy involve the migration of the micro SWMM model and numerous other ported mainframe models to 16-bit and 32-bit micro-based Unix and Xenix operating systems.

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Portability, Maintenance and FORTRAN Programing Style

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Introduction

At the Center for Water Quality Modeling, we have evolved a set of programing conventions that make our FORTRAN programs easily maintainable and transportable. These conventions have helped us in transporting many of our supported programs from minicomputers and mainframes to the microcomputer environment. This paper describes these programing conventions and development tools and discusses how they have aided in moving our programs to microcomputers while still providing a workable environment for the maintenance of code on different hardware and software systems.

Program Design Considerations

It is appropriate to discuss two aspects of programing style -- design considerations and coding conventions. Design relates to the overall structure of the program while coding conventions refer to the the actual FORTRAN implementation. Much has been written about structured programing and software design and we have found this design concept to be useful. The following paragraphs outline our approach to structured programing in FORTRAN.

Possibly the most important design consideration is modularity of structure. Modularity is fostered by writing small subroutines. To borrow a rule from the writing profession, be brief and concise. Treat a subroutine as you would a paragraph and confine it to a single thought -- a subroutine should only do one thing. A good rule is to restrict a subroutine to two pages or less; some prefer to limit a subroutine to a single page. But a single page may be too restrictive if your style uses a lot of open space to increase understanding. Always keep in mind that both you and others will have to read (and maintain) what you write.

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COMMON Blocks are a major part of our design. Although FORTRAN does not have code structures like many other languages, code structures such as a common work or data area can be emulated with COMMON Blocks. We also prefer that data be passed between subroutines using COMMON Blocks rather than Arguments, particularly at higher levels of the program. Although there are some arguable advantages to using arguments for this purpose (after all, that is their sole raison d'être), they can be troublesome. As discussed later, we have found them to be a frequent source of problems.

Of course, modular structure is much more than writing small subroutines and using COMMON Blocks. A clear specification of objectives and plan for achieving them is at the heart of structured programing. But these principles are a good beginning.

Although there may be good reasons for not considering it, portability is another important design consideration. Our programs are written to be used by others, mostly on their own computers and the programs have a surprisingly long lifetime. Although we cannot claim credit for the original design, QUAL2E (Brown and Barnwell, 1985) is based on a structure set out over 15 years ago. We use only 1977 ANSI Standard FORTRAN and avoid extensions to the standard.

If writing single-purpose code for a specific application, the programmer is justified in using the FORTRAN extensions available on his computer. In general, these extensions are provided to enhance the efficiency of both the programmer and the program. But if there is the slightest inkling that the program will be used on different hardware, the portability issue must be considered.

To insure portability, it is useful to identify target compilers that represent a reasonable spectrum of computers. For example, U S EPA staff can count on the availability of at least three kinds of computer -- IBM PC compatible, DEC VAX technology, and IBM 30xx-series mainframes. We run our code through compilers on each of these systems.

Where hardware-specific code must be used, the programmer must isolate and clearly identify this dependent code. For example, DATE and TIME subroutine calls are a common extension that are not part of the ANSI-77 standard but are available on many systems. Although not hardware-specific, file opening and closing conventions may vary among installations, even for the same type of hardware, and it is good practice to isolate these statements in a separate, clearly identified subroutine.

User Interface

The user interface is becoming more and more important to the computer programmer. The "friendliness" of programs in the microcomputer environment such as LOTUS 123™ have raised user expectations so high that the traditional batch-oriented card file input is fast becoming unacceptable. In fact, the Center for Water Quality Modeling has begun a major effort to develop a consistent interactive user interface for all our programs.

In designing a user interface, the programmer should strive to use complete English sentences for such things as error messages, status diagnostics, or run-time prompts. Although inside jargon and computerease may be expedient in the short run, the novice user will often be confused by such cryptic statements as "PROGRAM STOP - INPUT ERROR." Even an experienced programmer may wonder if this message was generated by the operating system or the program itself.

An important function of the user interface is verification of all inputs. Although the user interface may not be able to catch subtle errors in program input, several standard checks should be made. Perhaps the most obvious is checking of character versus numeric input. How many of us have not spent considerable time tracking down an error to find that its cause is a letter "O" where the number "0" should be, or vice-versa?

Another good practice is to explicitly check for real and integer numbers and to check that numbers are in a valid range. Also, default values should be stated or displayed explicitly. Cases exist where widely used computer programs with perhaps inappropriate default parameter values have inadvertently established "standard practice."

It is useful to place input verification criteria and defaults in tables or external files. This makes the criteria easy to locate and change. The use of an external file is particularly convenient as changes can be made without recompiling the program.

An echo of inputs should be provided as an option so that the user can document input when desired but switch it off when it is not needed. As most programs have the option of running either interactively or batch, the programmer should provide program traps to stop execution when input errors are uncovered rather than let execution continue.

And when writing tabular output from a program, concise but descriptive headings should be provided and output formatted for easy reading. Nothing is more frustrating than 10 pages of densely packed numbers with no identification. Include your documentation on the output, not in the users manual.

Documentation

Documentation is critical to a successful computer program. We think of documentation targeted both for the user and for the programmer. User documentation should include a description of the application or use of the computer program. The exact equations and solution techniques used as well as guidance for the user on ranges and typical values of input variables should be provided.

The actual installation of the program on the users' computer system, however, is often done by a programmer who is interested in quite different things. The programmer is interested in what we call "implementation documentation" that contains detailed instructions on how to install and maintain the program on a user's system.

Both the programmer and the user are interested in sample input and output to verify that the installation is correct and the program is operating correctly on their system. The system or implementation documentation also should include a functional organization of source code files by name as well as descriptions of program variables (particularly global variables) in case the user and his programmer have to troubleshoot the software. Often, the user may want to read the program in order to better understand what it is doing. This should be made as easy as possible by making available copies of the source code and identifying development and maintenance software.

Coding Conventions

Comments are an essential part of the source code and are often the only source code documentation. Although pseudocode is useful when designing a large program, we have found it difficult and costly to keep current. Each subroutine or program segment should have introductory comments that clearly state its purpose; describe inputs, outputs and modified variables; and identify error conditions. Comments are useful to physically separate sections of code that do not justify isolation in a subroutine and to explain loops and conditional statements. These are often the hardest parts of a FORTRAN program to decipher; particularly if the dreaded GO TO is used with indiscretion. (We all know the rule on GO TO -- DON'T!!!)

Complete English statements should be used; many programmers use a shorthand that can be completely indecipherable. Others will have to read what you write and base the maintenance and understanding of the source code on your commentary. When trying to decipher a program, remember that comments are not infallible -- they can lie.

Just as indentation and physical separation are used to help delineate parts of a composition, these devices should be used to graphically explain program logical flow and subordination. Loops should be indented and should end in the same column they begin. IF statements and IF-THEN-ELSE structures also should follow a consistent indentation format. Comments may be helpful in separating parts of loops and conditional statements.

Indentation is useful when it is necessary to continue COMMON, DATA and other type and declaration statements beyond a single line. For example, beginning the continuation lines at, say, column 10 helps to delineate blocks of code. And if there is no alternative to using a GO TO construct (remember the old maxim -- rules are for privates and lieutenants; experienced programmers know when to break the rules), indentation is useful to help identify the continuations.

Modularity and structure in code can also be enhanced by placing COMMON Blocks in separate source code files that may be brought into other source code files via FORTRAN's INCLUDE statement. In addition to producing smaller subroutines, use of INCLUDE files for COMMON blocks will insure that COMMONs are consistent across a program. Don't dimension or name variables differently in different subroutines. If it is necessary to rename variables, use the EQUIVALENCE statement.

It also is useful to include data type specifications in INCLUDE files. If possible, group different type variables (eg, CHARACTER, INTEGER and REAL) into separate COMMON blocks. This practice will help avoid boundary alignment problems and enhances the maintainability of a program. Explicit declarations of INTEGER*2 and INTEGER*4 data type are also helpful. Most computers default to the INTEGER*4 data type but, on microprocessors in particular, the INTEGER*2 calculations are considerably faster than INTEGER*4 arithmetic (Microsoft, 1985) in addition to saving some memory.

We have found it desirable to isolate file OPEN and CLOSE statements in a central, clearly identified routine because, as mentioned above, these statements may be installation-dependent. It also is good practice to use variables for unit numbers in input-output statements and assign these variables in an "environment" INCLUDE file, or within in the file-open subroutine. Always open terminal input-output units explicitly. Although some installations assume units 5 and 6 for terminal input and output, respectively, others will use unit 5 for both and still others may use units 1 and 2.

Where possible, the user should be allowed to specify file names either at execution or at run time rather than specify the names in the compiled code. This will allow easier installation of code. In addition, the OPEN statement error return codes should be used to recover open errors gracefully, giving explicit error messages using complete English statements.

We favor the use of the PARAMETER statement or a BLOCK DATA subroutine to declare array dimensions for system-wide variables, DO loops, I/O units and other appropriate system wide parameters. Proper use of the PARAMETER statement will allow easy modification of program limits by modifying a single subroutine or source code file brought into the program with an INCLUDE file. This feature is useful if one wishes to modify a program for different machine memory configurations. For example, the IBM/PC version of the QUAL2E program is distributed with array dimensions set for a 256K machine. These array dimensions can be changed by modifying one line in a single subroutine. One should be careful, however, not to use PARAMETER to set values for variables. Remember that the program source code must be edited, recompiled, and linked, to change any PARAMETER or a variable initialized in a BLOCK DATA subroutine.

In defining variables, explicit declarations of variable type should be used rather than the default type. A useful extension on some compilers is the IMPLICIT NONE statement. This statement will force explicit declaration of all variables when writing code and can be easily removed for the distribution version. Dimensioning variables in INTEGER and REAL Statements was considered good practice but we have found this to be a problem with microcomputer compilers that only allow dimensioning in the DIMENSION and COMMON statements and require type specification in separate statements. As mentioned above, the same name should be used everywhere. This helps avoid later confusion when debugging the code. Prefix/Suffix-Type variable names are also useful in helping readability. And a clear specification of initial value avoids problems with systems where a zero default initial value is not the convention.

Downloading Problems

In the process of downloading our software to the microcomputer, we have encountered several problems that relate to programing style and coding conventions discussed above. In general, microcomputer FORTRAN compilers are quite strict in following the ANSI standard for FORTRAN 77. Few if any FORTRAN language extensions are available on microcomputers and practices that are acceptable on larger computers are not acceptable with these compilers.

For example, the compilers are quite sensitive to statement order and will not allow mixing of specification, declaration and executable statements. Variables that are assigned character string values should be defined as Character in a type specification. Programmers should be careful not to use variable names longer than six characters as the more restrictive compilers will not recognize differences beyond the sixth character.

A good practice is to separate character and numeric data into individual COMMON blocks to avoid problems with boundary alignment. The same is true of integer, real and double precision variables. And one should avoid mixing these different type variables in executable statements, always using explicit type conversions in arithmetic statements.

Another good practice is to initialize variables in a BLOCK DATA subroutine and avoid reinitializing with DATA statements. On micros, argument types must be compatible in CALL and SUBROUTINE statements. Although compilers on larger computers may convert between types, the microcomputer compilers do not.

Some compilers will allow FORMAT statement field separators to be omitted -- on micros, this will cause errors. ENCODE and DECODE statements should not be used; they have been replaced by internal READ and WRITE statements in FORTRAN 77.

Development Tools

There are a number of FORTRAN compilers available for personal computers. Two recent reviews in PC Tech Journal (Howard, 1985) and Computer Language (Bensor et al., 1986) discussed nine different FORTRAN compilers (Table 1) for the MS-DOS operating system and the Macintosh.

Several letters responding to the Howard's review are contained in the March 1986 issue and provide additional insight to the compilers. Of particular interest is an editorial in the March 1986 PC Tech Journal (Fastie, 1986) that observes "the volume of comment on our coverage indicates considerably larger interest in FORTRAN than expected ... confirming that FORTRAN is alive and well. ... One particularly surprising comment is about FORTRAN's portability. Because it is not a systems programmers language, programmers are not as likely to exploit extensions supplied by a particular vendor, opting instead for a more standard approach. This portability extends beyond the desktop to the minicomputers and mainframes crunching FORTRAN."

TABLE 1. REVIEWS OF PC FORTRAN COMPILERS

Compiler	PC Tech Journal Oct. 85	Computer Language Jan. 86
IBM Professional FORTRAN (a.k.a. Ryan-McFarland)	X	X
Lahey	X	X
Microsoft	X	X
Digital Research	X	X
Intel FORTRAN-86		X
Prospero ProFORTRAN		X
Supersoft		X
WATFOR-77		X
Microsoft MACFORTRAN		X

The Computer Language article is particularly interesting because it also discusses a number of add-on tools that provide graphics, string manipulation, windows, scientific subroutines, and productivity enhancements.

Our experience is limited to the Microsoft™ and IBM Professional™ (a.k.a. Ryan-McFarland™) FORTRAN (ProFORT) compilers. Microsoft's compiler is flexible in that it can produce executable code that executes on PCs without the 8087 Math Coprocessor chip, whereas ProFORT requires the coprocessor. However, Microsoft's FORTRAN syntax, particularly for INCLUDE statements, and use of Metacommands for compiler options differ from conventional usage.

We have found that ProFORT is compatible with our primary development environment, DEC VAX FORTRAN. ProFORT is a full implementation of FORTRAN 77 with few, if any, extensions and is often 100% compatible with other FORTRAN development environments. This compatibility has been a key to our ability to develop and maintain software targeted for a wide variety of computer systems.

The key to installing large FORTRAN programs on the PC lies in the linkage editor. The LINK command provided with both the Microsoft and ProFORT compilers will allow only one level of overlay and will only allow overlay of code segments. In trying to link a large program (25,000 lines of code) with these linkage editors, we received a message indicating that code segment DGROUP exceeded 64K (Parker, 1986, discusses this problem). Phoenix Software's PLINK86 linkage editor allowed us to bypass this problem by allowing multiple overlay levels (up to 32 levels of overlay may be used). A complex overlay structure may be required by large program size and proper software design can make for a much more efficient overlay structure.

Summary

We have reviewed programing design and coding conventions that have served us well in maintaining and transporting out FORTRAN programs among a variety of computers. The development tools discussed above make it possible to install quite large FORTRAN programs on microcomputers and our experience is showing that the microcomputer can be an attractive environment for running these programs. When execution times are excessive, the programs can be easily moved to larger machines to take advantage of their faster speed.

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CONSIDERATIONS ON USE OF MICROCOMPUTER MODELS FOR STORMWATER MANAGEMENT

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ABSTRACT

Widespread use of microcomputer hydrologic models raises the question of their optimum design. Microcomputer models developed in North America and Europe can be classified into two categories:

1. Models with explicit incorporation of many parameters which can be used as default values or modified for calibration purposes if data is available.

2. Models reducing drastically the number of parameters and incorporating default values in the program without giving the user a possibility for a change. In this category rainfall distributions are also built in.

The paper gives examples of possible sources of errors associated with the second approach. If relatively sophisticated techniques are used to simulate rainfall runoff processes, these methods can be incorporated in a microcomputer user friendly model and still provide enough flexibility to allow non-modellers to modify default parameters in cooperation with a specialist. This also provides a better understanding of hydrologic principles. The paper also describes a Canadian package of microcomputer models based on this principle and some implementation aspects. A practical application is also described. The main runoff simulation techniques presented in this paper give consistant results with detailed SWMM for urban areas and with appropriate selection of parameters results can be compatible with SCS type simulations for rural areas.

1. INTRODUCTION

In the early 70's the profession started to consider urban hydrologic models as a desirable replacement for empirical methods for runoff control, such as the Extended Rational Method. The use of models was advocated mainly because they can be tested against rainfall-distributions, real or synthetic. This objective can be achieved using various principles of rainfall-runoff transformation such as kinematic routing, parallel non-linear reservoirs, single quasi linear reservoirs, etc. A discussion of the advantages or disadvantages of each type of model and mainly of "routing versus convolution", is beyond the objective of this paper. What all these models have in common is that the user can modify several parameters for calibration and verification of applicability for specific conditions. This also applies for rainfall loss models such as Green-Ampt, Horton, Holtan, etc.

Flexibility in selection of parameters or use of various storm distributions is an obvious advantage for a hydrologist, but may be confusing for a practising drainage engineer. Calibration is not always possible because of lack of data. Under these conditions two approaches are used in practical applications:

A. Models in which a relatively large number of parameters can be used either as default values or modified for calibration purposes.

B. Simplified models with default values incorporated in the program and very few input parameters.

Examples of packages of microcomputer programs in the second category are those based on the SCS Curve Number method with an empirical relation for the initial losses (ie: $Ia = 0.2S$). Runoff is computed by means of the SCS curvilinear unit hydrograph with empirical relations for the response time incorporated in the program.

These models can be even simpler if the rainfall pattern is already built in (Debo, 1985). Attempts of simplification are also made in a more sophisticated model used in France (Chocat, 1984). Although it has interesting features, runoff contribution from the pervious area in urban watersheds is neglected, which is not applicable for North American conditions.

Although the response of urban watersheds has a higher degree of nonlinearity as compared to rural areas, microcomputer simplified models based on SCS techniques, Santa-Barbara method, etc., assume linearity in both cases.

In most recent microcomputer developments a significant effort is made only towards user friendly structures, attractive output displays and extensive graphical capabilities. Although these are important features they will not be discussed in this paper, which is mainly intended to advocate the advantages of A (above). This approach is possible without reducing the ease of use.

2. SIMPLICITY versus ACCURACY

The pitfalls of oversimplifications can be illustrated by Canadian experience with the SCS methodology (SCS, 1971).

In the SCS procedures, relation (1) in figure 1 between total rainfall and runoff, has only two parameters; soil storage and initial abstraction, Ia. The initial abstraction is related to the soil storage by the empirical relation Ia = 0.2S.

The SCS unit hydrograph is automatically defined by the response time which is related to the time of concentration by an empirical relation.

By means of these simplifications, the practising engineer has to specify the same number of parameters as in the Rational Method, namely;

1. The soil storage or the CN number which is equivalent to a runoff coefficient.
2. The time of concentration.

Comparisons of simulations and measurements conducted by the IMPSWM (Implementation of Storm Water Management) Program at the University of Ottawa (Wisner et. al., 1984) have shown that the cascade of linear reservoirs unit hydrograph proposed by Nash (Nash, 1957), has an adequate performance for small rural watersheds. This unit hydrograph is given by a gamma function defined with two parameters: n, the number of reservoirs or shape factor and t_p, the time to peak (relation(2), figure 1). By proper selection of n and t_p several shapes of unit hydrographs can be reproduced. For instance the shape of the SCS unit hydrograph is a particular case of the gamma function for n = 4.75.

Table 1 shows the values of n determined after calibration of the Nash model for 6 rural areas (Wisner et. al., 1984). These results were obtained using a modified SCS curve number method in which the initial abstraction is an input (Ia<>0.2S). It can be seen that the values of the shape factor n are less than 4.75, the value corresponding to the SCS unit hydrograph. For similar conditions a default value of n = 3 seems to be more adequate.

The initial abstraction was also found to vary from 1 to 4 mm; this is less, by orders of magnitude than the value given by the default value, Ia = 0.2S. It was also verified that for some of these watersheds the use of more sophisticated infiltration methods such as Green-Ampt or Holtan did not significantly improve the results (Consuegra et. al., 1984).

The Nash model was also compared with a more sophisticated variable unit hydrograph (VUH, Ding, 1974). Results also showed that after calibration the results with the Nash model and the non linear model were not very different. Based on the satisfactory performance of the Nash model and of the modified Curve Number procedure, it was decided to incorporate this

Table 1. Comparison of calibrated values of n and tp for 6 rural watersheds with the Nash Model. Comparison of calibrated tp and tp computed using Williams equation.

Watershed	Location	Area ha	Slope %	Tp Calibrated hrs	Tp Williams hrs	n
Wixon Creek	Ontario	1016	1.2	3.7	1.42	2.6
Holiday Creek	Ontario	3053	0.7	5.7	2.82	2.0
North Morrow	Ontario	382	1.0	6.0	3.31	2.0
Clément Ménard	Ontario	96	0.1	2.0	1.93	1.85
Collins Creek	Ontario	15500	0.15	69.0	11.8	3.0
Parimbot	Switzerland	380	2.0	1.7	0.61	3.0

Williams formula: $Tp = 6.54\ A^{0.39}\ S^{-0.50}$

Tp = hours

A = surface sq. m.

S = slope ft/mile

technique in the microcomputer programs described in the next sections of this paper. The resulting submodel is called NASHYD.

The SCS procedures initially developed for rural areas were later recommended for urban areas (SCS TR55). IMPSWM studies for urban watersheds were conducted by first comparing, for a hypothetical watershed, various linear and quasi linear models with EPA.SWMM (Huber et. al., 1982) simulations under design conditions. It was found that the performance of some single linear or quasi linear reservoirs could give results compatible with EPA.SWMM for a limited range of conditions (high imperviousness, etc.). Very good consistency with SWMM was obtained with a two parallel quasi linear reservoir conceptual model in which the storage coefficient is a function of the time of equilibrium (P'ng, 1982). The resulting URBHYD model based on this principle uses relation (3) in figure 1, as proposed by Pedersen et. al., (1980). URBHYD was also compared with measurements on 4 urban catchments with good results as shown in table 2. (Wisner and Consuegra, 1986). Flows shown in table 2 were obtained with SWMM default values. Later, testing studies on European watersheds showed the need for slight modifications of default parameters specially for low rainfall volumes.

The selection of meteorological input is also a controversial issue. It is useful to compare results from design storms and series of historical storms. Studies conducted by the IMPSWM program (Wisner and Fraser, 1982) compared Chicago (Keyfer and Chu, 1957) and SCS (type 2, 24 hours) design storms with a series of real intense storms in the Metro-Toronto area. Peak flows were computed with URBHYD and NASHYD for urban and rural areas respectively. For rural areas the Curve Number (CN) was obtained from a correlation with API (Antecedent Precipitation Index).

For urban areas it was found that Chicago design storms generated peak flows closer to observed values than the SCS storms (fig. 2).

For rural areas, flows are strongly dependent on moisture conditions and infiltration losses. With an appropriate selection of CN, the SCS storms performed better than the Chicago design storms. For urban areas and under design conditions, peak flows obtained with SWMM and URBHYD were found to be very sensitive to the maximum rainfall intensity. For the Metro Toronto region, it was found that a time step of 10 minutes for the Chicago design storm gives realistic peak flows.

The above examples show that microcomputer models used for comparison of pre and post-development conditions can not be excessively simple. If simplifications are introduced they should be tested with measurements for a variety of conditions and their limitations clearly defined. As an example, the default value Ia = 0.2S may be acceptable for high rainfall volumes. A single quasi linear reservoir can give good results for high degrees of imperviousness. Simplified traditional methods can be applied mainly for small areas. Simplified microcomputer models should also give the user the possibility to compare synthetic with real storms.

HYDROLOGICAL SUBMODEL	METHOD	EQUATIONS	DEFAULT VALUES
URBHYD Urban Runoff	Horton equation for losses	$f_t = f_c + (f_o - f_c)\ e^{-k.t}$	f_o = 76.2 mm hr
		f_t = infiltration rate at time t	f_c = 13.2 mm hr
		f_o = initial infiltration rate	k = 4.14
		f_c = final infiltration rate k = dekay factor	
	Two parallel single linear reservoirs for pervious and impervious areas	For both reservoirs, Storage coefficient K	
		$K = a. \frac{L^{0.6}\ n^{0.6}}{i^{0.4}\ S^{0.3}}$ (3)	$n_{pervious}$ = 0.25
		a = conversion factor L = length of watershed	$n_{impervious}$ = 0.013
		n = manning surface roughness	$L = (1.5\ Area)^{0.5}$
		i = maximum rainfall intensity	
		S = slope of the watershed	
NASHYD Urban Runoff	Modified SCS technique	$Q = \frac{(P-Ia)^2}{P-Ia+S}$ (1)	
		Q = runoff (mm) P = precipitation (mm)	
		S = soil water storage Ia= initial abstraction (mm)	Ia<0.2.S.Ia = 3 mm
		$CN = \frac{25400}{254+S}$ (1)	
		CN= modified curve number related to Antecedent Precipitation index (API)	
	Nash Cascade of linear identical reservoirs	Unit hydrograph defined by two parameters,	
		n = shape parameter	n = 3
		T_p = time to peak of unit hydrograph	
		$q = qp\ (t/T_p)^{n-1}\ e^{(1-n)((t/T_p) - 1)}$ (2)	
		qp= unit hydrograph peak	

Fig. 1 Description of NASHYD and URBHYD

Table 2. Comparison of observed and simulated peak flows with lumped OTTHYMO and detailed SWMM for 4 urban catchments

Gray Haven

Storm date	Observed peak (cms)	Simulated peak OTTHYMO (cms)	Simulated peak SWMM (cms)
05-06-63	2.29	2.05	2.21
10-06-63	2.24	2.05	2.21
14-06-63	0.87	0.89	0.89
20-06-63	0.84	0.90	0.97
01-08-63	2.49	1.86	1.79
14-08-63	0.98	1.19	1.09
02-08-66	0.86	0.93	0.98

Hillbrow

Storm date	Observed peak (cms)	Simulated peak OTTHYMO (cms)	Simulated peak SWMM (cms)
21-01-78	2.97	3.41	2.78
16-12-83	4.62	4.53	4.62
30-12-83	2.54	2.46	2.18
01-01-84	4.43	4.41	5.38

Malvern

Storm date	Observed peak (cms)	Simulated peak OTTHYMO (cms)	Simulated peak SWMM (cms)
22-09-73	0.92	0.74	0.98
23-09-73	0.71	0.46	0.63
31-05-74	0.90	0.85	1.11
21-06-74	0.58	0.56	0.73

Pinetown

Storm date	Observed peak (cms)	Simulated peak OTTHYMO (cms)	Simulated peak SWMM (cms)
13-02-79	1.20	1.22	1.11
22-05-79	1.07	0.74	0.72
18-02-80	1.59	1.35	1.34

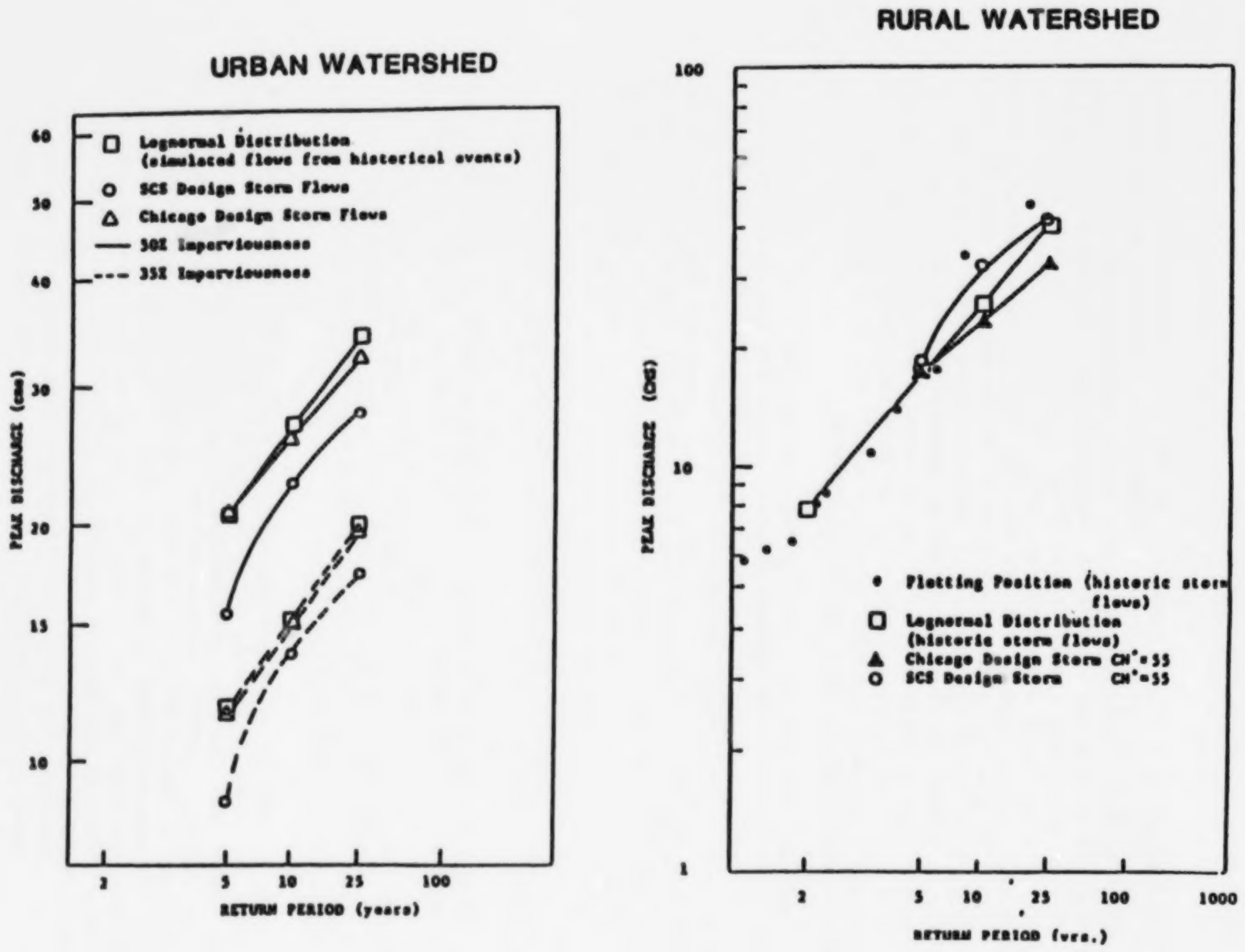


Figure 2. Comparison of design storms and a series of real storms for Metro-Toronto conditions

3. USER FRIENDLINESS versus FLEXIBILITY

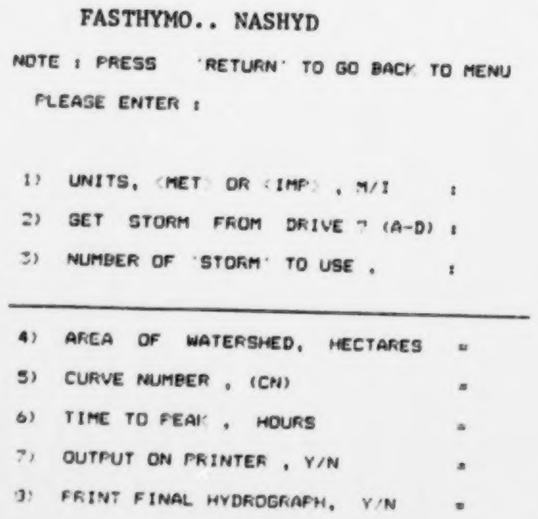
An example of a simple procedure for the incorporation of flexibility in a user friendly program is the FASTHYMO model. FASTHYMO was developed by a consulting firm participating in the IMPSWM program (Brodie et. al.,1984) FASTHYMO can be used for simplified lumped watershed simulation with URBHYD and NASHYD, using very simple inputs (figs. 3 and 4). The user of FASTHYMO works with a series of design storms, given by municipal criteria or with real storms. Once these storms are entered, they can be permanantly stored on disk. The user can first check list of storms available on disk. Prior to simulations, users can review all default values stored on disk and eventually modify and store a new set of parameters on disk. Figure 5 displays a typical parameter file. As an example for rural areas a default value of $n = 3$ and a value of $Ia = 1.5$ mm were recommended for Metro Toronto conditions. The user can also specify $n = 4.75$ and $Ia = 0.2S$ to ensure complete compatibility with the SCS procedures.

However, the user must specify the time to peak of the Nash unit hydrograph (fig. 3). No attempt was made to include one of the available formulas for t_p. It was felt that this may limit the choices of the user and promote a given empirical formula without testing it. Table 1 shows a comparison between calibrated values of t_p and computed ones using the well known Williams formula (Willians and Hann, 1973) for the same rural areas. It is evident that differences may be significant. Similar discrepancies may be found using other relations. An analysis of travel time (overland and channel), preferably by hand computation, is recommended. Providing t_p as an input also allows for sensitivity analysis and a better understanding of the model. The rest of the input data are very easy to determine (fig. 3).

Figure 5 also shows a set of default parameters used to compute runoff from urban areas. The meaning of these parameters was already presented in fig. 1. A default relation between length and area is also provided. For maximum flexibility, the user can also specify particular values for the storage coefficient and use the model as a linear one. If a municipality intends to standardize parameters it can use values from calibration studies or those recommended by a specialist.

For a complete watershed analysis including detention reservoirs channel and conduit routing, users require the more advanced models described further on. The same principles were maintained for parameters and design storms. PLANHYMO (Brodie and et. al.,1984) is a simple interactive microcomputer watershed model for simplified watershed simulation. The input for each watershed is similar to that of FASTHYMO. At each junction the user specifies all the information related to the different sub-areas. He also indicates the type of routing (if any) to the next downstream junction. PLANHYMO has the following capabilities:

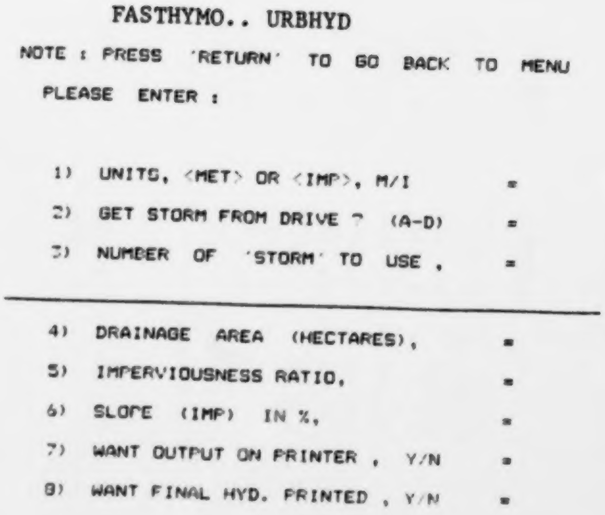
1. Determination of runoff hydrographs from contributing urban and rural areas at each junction with URBHYD and NASHYD, respectively.



```
     FASTHYMO.. NASHYD
NOTE : PRESS   'RETURN' TO GO BACK TO MENU
  PLEASE ENTER :

 1)  UNITS, <MET> OR <IMP> , M/I        :
 2)  GET  STORM  FROM  DRIVE ? (A-D) :
 3)  NUMBER OF 'STORM' TO USE .         :
______________________________________________
 4)  AREA  OF  WATERSHED,  HECTARES     =
 5)  CURVE NUMBER , (CN)                =
 6)  TIME TO PEAK ,  HOURS              =
 7)  OUTPUT ON PRINTER , Y/N            =
 8)  PRINT FINAL HYDROGRAPH,  Y/N       =
```

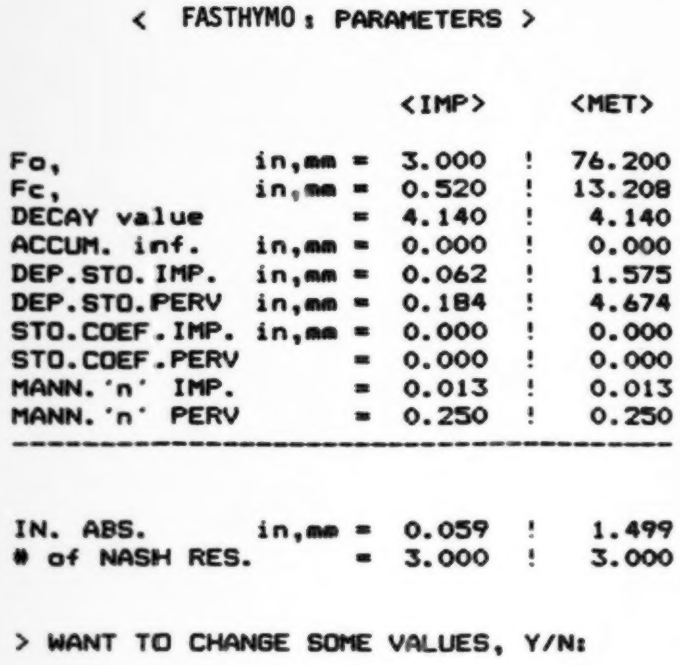
Figure 3. Window display for NASHYD in FASTHYMO



```
       FASTHYMO.. URBHYD
NOTE : PRESS  'RETURN'  TO  GO  BACK  TO  MENU
  PLEASE  ENTER :

  1)  UNITS, <MET> OR <IMP>, M/I          =
  2)  GET STORM FROM DRIVE ?  (A-D)       =
  3)  NUMBER  OF  'STORM' TO  USE ,       =
______________________________________________
  4)  DRAINAGE  AREA  (HECTARES),         =
  5)  IMPERVIOUSNESS RATIO,               =
  6)  SLOPE  (IMP)  IN %,                 =
  7)  WANT OUTPUT ON PRINTER ,  Y/N       =
  8)  WANT FINAL HYD. PRINTED , Y/N       =
```

Figure 4. Window display for URBHYD in FASTHYMO



```
        < FASTHYMO : PARAMETERS >

                             <IMP>     <MET>

Fo,              in,mm =  3.000  !  76.200
Fc,              in,mm =  0.520  !  13.208
DECAY value            =  4.140  !   4.140
ACCUM. inf.      in,mm =  0.000  !   0.000
DEP.STO.IMP.     in,mm =  0.062  !   1.575
DEP.STO.PERV     in,mm =  0.184  !   4.674
STO.COEF.IMP.    in,mm =  0.000  !   0.000
STO.COEF.PERV          =  0.000  !   0.000
MANN.'n' IMP.          =  0.013  !   0.013
MANN.'n' PERV          =  0.250  !   0.250
------------------------------------------


IN. ABS.         in,mm =  0.059  !   1.499
# of NASH RES.         =  3.000  !   3.000

> WANT TO CHANGE SOME VALUES, Y/N:
```

Figure 5. Parameter file in FASTHYMO

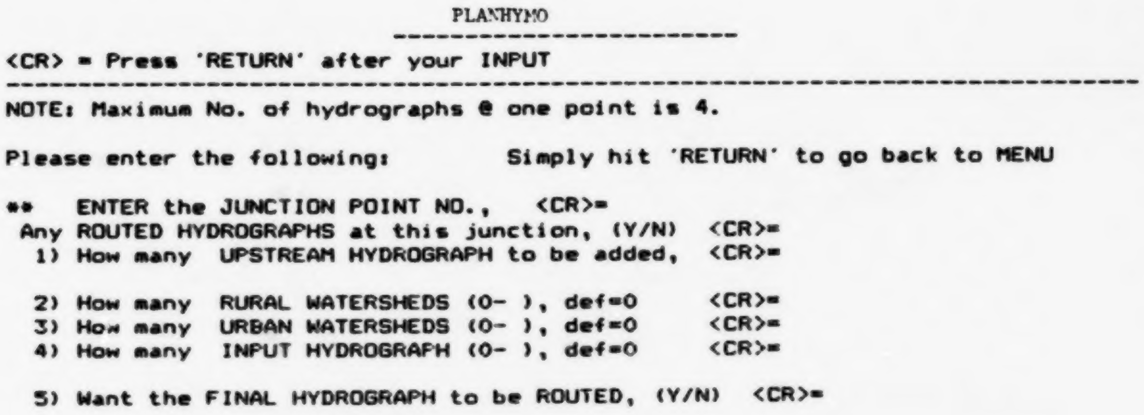
2. Addition of these hydrographs with other input hydrographs eventually determined from other computations.

3. Routing of hydrographs through reservoirs, channels and pipes.

4. Storage or printing of summary results.

Figure 6 shows a typical input display of PLANHYMO.

More recently, LOTHYMO (Consuegra et. al., 1986) was developed as an extension of PLANHYMO. The structure of LOTHYMO has been developed to meet the following criteria: fully interactive input and output processes and secondly, efficient executional performance. The first objective was achieved by interfacing PLANHYMO with the powerful spread-sheet, LOTUS 1-2-3. Using ready made templates, input files can be easily created. The systematic tabulation of all input parameters permits rapid entry, quick review and ,therefore, quick detection of any inconsistencies. In the output stage, results can be quickly retrieved by LOTUS 1-2-3 for their review, interpretation and management. Graphics can also be generated automatically. The second objective was satisfied by the development of a driver program written in BASIC, which reads the input files created with LOTUS and calls all the appropriate hydrological sub-routines for the entire drainage network. At this stage absolutely no user interaction is required.



```
                           PLANHYMO
                    -----------------------
<CR> = Press 'RETURN' after your INPUT
---------------------------------------------------------------------------
NOTE: Maximum No. of hydrographs @ one point is 4.

Please enter the following:        Simply hit 'RETURN' to go back to MENU

**   ENTER the JUNCTION POINT NO.,    <CR>=
 Any ROUTED HYDROGRAPHS at this junction, (Y/N)  <CR>=
  1) How many  UPSTREAM HYDROGRAPH to be added,  <CR>=

  2) How many  RURAL WATERSHEDS (0- ), def=0     <CR>=
  3) How many  URBAN WATERSHEDS (0- ), def=0     <CR>=
  4) How many  INPUT HYDROGRAPH (0- ), def=0     <CR>=

  5) Want the FINAL HYDROGRAPH to be ROUTED, (Y/N)  <CR>=
```

Figure 6. Window display for PLANHYMO

Figure 7 shows the Highgate Creek (Ontario) watershed where LOTHYMO was applied. The total area is 540 ha. The downstream area is urbanized and drained by an open channel with a capacity of 16.4 cms. In the near future, the entire watershed will be urbanized. If no runoff control measure is undertaken, the 100 year post-development flow will raise up to 42.6 cms. Three detention reservoirs are proposed to reduce the post-development flow. Schematization for the post-development conditions is also shown in figure 7. Figures 8 and 9 show the output obtained with LOTHYMO and the corresponding plots generated with LOTUS.

4. EXPERIENCE WITH THE MODELS

FASTHYMO and PLANHYMO are presently used on IBM PC and Apple IIe for quick project verification by organizations such as the Department of Engineering of the Town of Markham, the Department of Public Works of the City of Scarborough, the Project Review and Approval Group of the Ontario Ministry of Environment, the Water Resources Service of the Metro Toronto Conservation Authority, etc.

The following are examples of projects using the flexible microcomputer models presented in this paper, illustrating the range of applications by consultants:

i) Preliminary Master Drainage Plans. A consulting firm in Montreal applied PLANHYMO for an area including two dry ponds and three wet ponds. The application was done during a two day training session on the job with an engineer without previous computer experience.

ii) Safety study for a SWM reservoir. A consulting firm in Illinois used PLANHYMO for the analysis of a reservoir during the maximum possible storm.

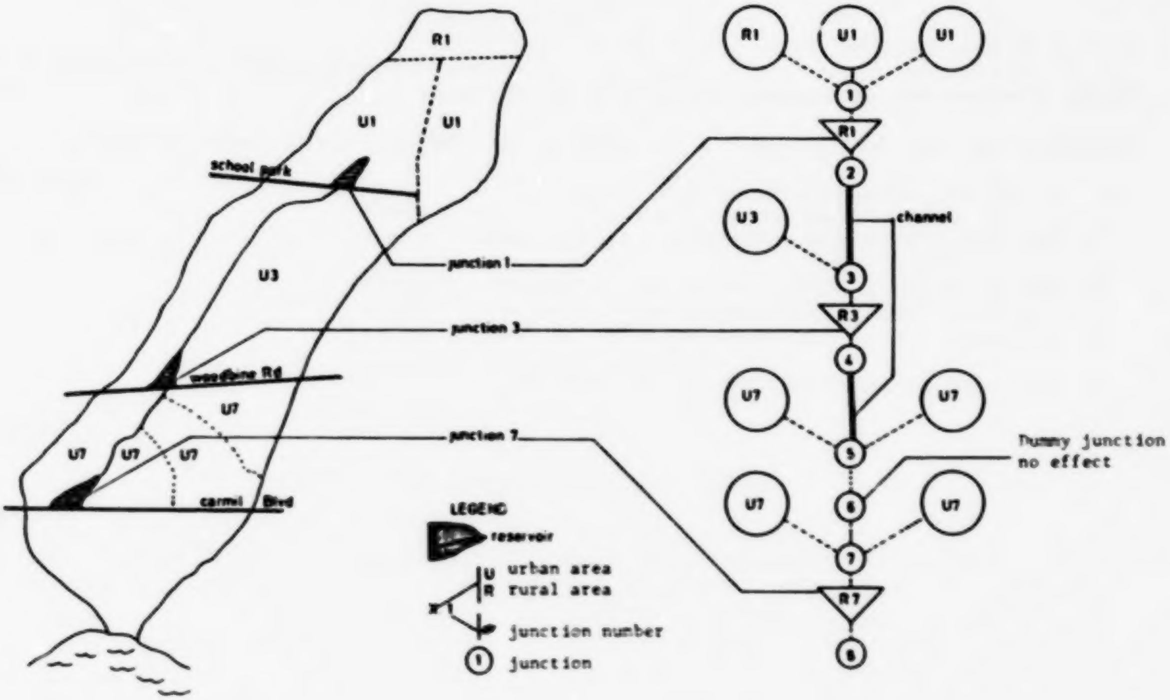

R1
U1
U1
school park
junction 1
U3
junction 3
woodbine Rd
U7
U7
U7
U7
junction 7
carmil Blvd
LEGEND
reservoir
U urban area
R rural area
junction number
① junction
R1
U1
U1
1
R1
2
U3
channel
3
R3
4
U7
U7
5
Dummy junction
no effect
U7
6
U7
7
R7
8

Figure 7. Highgate Creek watershed and post-development schematization

iii) Multiple on-site detention reservoirs for an industrial park. A consulting firm in Ottawa designed a drainage system with three storages for large parking lots and a separate storage for the minor system with PLANHYMO. The objective of this study was to drastically reduce peak flows due to a lack of downstream capacity.

Drainage engineers involved in the review of projects presented above, did not have previous experience with hydrologic modelling and initially needed indications regarding the choice of default values. Two years of experience with these microcomputer models show that practising engineers can easily modify the described microcomputer models to suit their own needs. Some of them perform sensitivity analyses on their own initiative. Storms have also been compared, and this led to useful discussions.

5. FINAL COMMENTS

As indicated in the introduction, there are several models for rainfall-runoff transformation, infiltration losses and design storms. While their advantages and disadvantages are still discussed by hydrologists, there is some agreement, that with proper parameters reasonable results can be

```
                    SUMMARY OUTPUT FOR NETWORK
                    --------------------------

           SUMMARY OUTPUT FOR JUNCTION POINT NO. 1
           ---------------------------------------

WATERSHED      AREA    IMP    CN*    S      PEAK    TPEAK    RUNOFF VOL
               ha.     %             %      cms     hrs          mm
 Rural         47.21   **     61     1.2    1.39    13.00       42.76
 Urban         60.53   27     **     1.3    9.24    11.00       51.65
 Urban         57.50   70     **     1.6   14.27    11.00       84.45
 TOTAL=       165.24   **     **     **    23.09    11.00       60.53
RROUTE TO  3  165.24   **     **     **     4.65    12.00       60.52
           SUMMARY OUTPUT FOR JUNCTION POINT NO. 2
           ---------------------------------------

WATERSHED      AREA    IMP    CN*    S      PEAK    TPEAK    RUNOFF VOL
               ha.     %             %      cms     hrs          mm
U/S JN No 1   165.24   **     **     **     4.65    12.80       60.52
 TOTAL=       165.24   **     **     **     4.65    12.80       60.52
        to 3  165.24   **     **     **     4.64    13.16       60.50
           SUMMARY OUTPUT FOR JUNCTION POINT NO. 3
           ---------------------------------------

WATERSHED      AREA    IMP    CN*    S      PEAK    TPEAK    RUNOFF VOL
               ha.     %             %      cms     hrs          mm
 Urban        213.40   30     **     1.1   27.62    11.00       53.93
U/S JN No 2   165.24   **     **     **     4.64    13.16       60.50
 TOTAL=       378.64   **     **     **    30.92    11.00       56.80
RROUTE TO  4  378.64   **     **     **    10.30    12.80       56.80
           SUMMARY OUTPUT FOR JUNCTION POINT NO. 4
           ---------------------------------------

WATERSHED      AREA    IMP    CN*    S      PEAK    TPEAK    RUNOFF VOL
               ha.     %             %      cms     hrs          mm
U/S JN No 3   378.64   **     **     **    10.30    12.80       56.80
 TOTAL=       378.64   **     **     **    10.30    12.80       56.80
        to 5  378.64   **     **     **    10.30    13.24       56.80
```

```
           SUMMARY OUTPUT FOR JUNCTION POINT NO. 5
           ---------------------------------------

WATERSHED      AREA    IMP    CN*    S      PEAK    TPEAK    RUNOFF VOL
               ha.     %             %      cms     hrs          mm
 Urban         26.70   30     **     0.8    4.44    11.00       53.94
 Urban         44.00   30     **     0.8    6.04    11.00       53.94
U/S JN No 4   378.64   **     **     **    10.30    13.24       56.80
 TOTAL=       449.34   **     **     **    15.88    11.00       56.34
           SUMMARY OUTPUT FOR JUNCTION POINT NO. 6
           ---------------------------------------

WATERSHED      AREA    IMP    CN*    S      PEAK    TPEAK    RUNOFF VOL
               ha.     %             %      cms     hrs          mm
U/S JN No 5   449.34   **     **     **    15.88    11.00       56.34
 TOTAL=       449.34   **     **     **    15.88    11.00       56.34
           SUMMARY OUTPUT FOR JUNCTION POINT NO. 7
           ---------------------------------------

WATERSHED      AREA    IMP    CN*    S      PEAK    TPEAK    RUNOFF VOL
               ha.     %             %      cms     hrs          mm
 Urban         34.00   30     **     0.8    5.58    11.00       53.94
 Urban         39.70   30     **     0.8    6.26    11.00       53.94
U/S JN No 6   449.34   **     **     **    15.88    11.00       56.34
 TOTAL=       523.04   **     **     **    27.72    11.00       56.00
RROUTE TO  8  523.04   **     **     **    15.07    12.60       56.00
           SUMMARY OUTPUT FOR JUNCTION POINT NO. 8
           ---------------------------------------

WATERSHED      AREA    IMP    CN*    S      PEAK    TPEAK    RUNOFF VOL
               ha.     %             %      cms     hrs          mm
U/S JN No 7   523.04   **     **     **    15.07    12.60       56.00
 TOTAL=       523.04   **     **     **    15.07    12.60       56.00
```

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best available copy.

Figure 8. Output generated with LOTHYMO for the Highgate Creek watershed

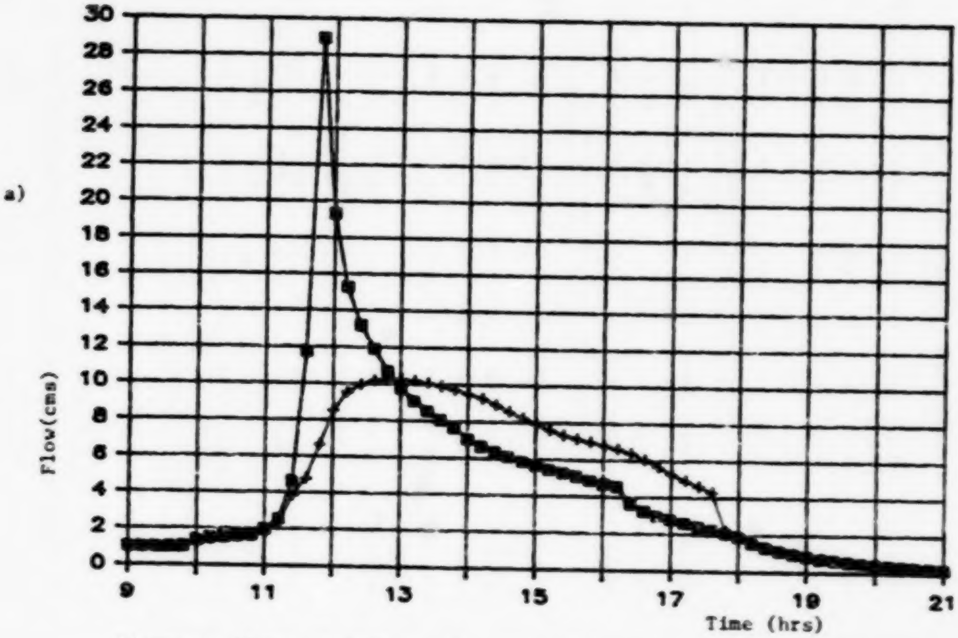
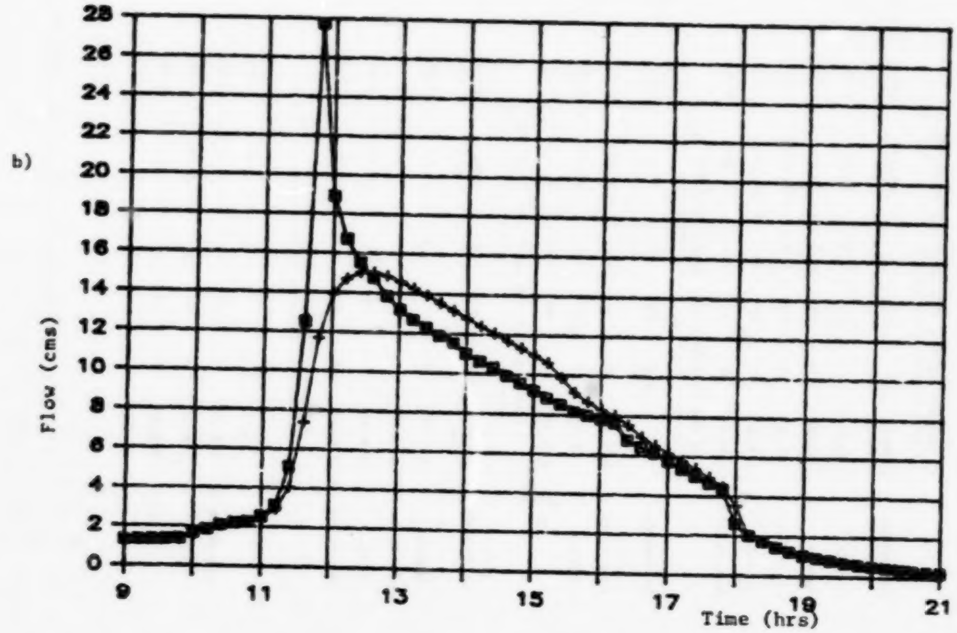


Figure 9. Plots of inflow and routed hydrograph generated with LOTUS at junction 3 (a) and at junction 7 (b)



obtained with several types of models. A key issue in Storm Water Management is whether the rapidly developing microcomputer software for practising drainage engineers should or should not promote a better understanding of basic hydrologic principles.
It is not yet known which of these options will prevail. The two years of experience with some of the models discussed above, proves, that at least the first approach is possible.

Many agencies require consultants to use standardized parameters, a given unit hydrograph and a specific design storm. Experience with the models proves that even in these situations there is some advantage in comparing results obtained with standardized parameters and other input parameters.

Information on all models based on the IMPSWM research is available from The IMPSWM group, Department of Civil Engineering, University of Ottawa, Ottawa, Canada, telephone (613) 564-3911.

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PC SOFTWARE FOR COMPUTATIONAL HYDROLOGY

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ABSTRACT

The Computational Hydraulics Group at McMaster University has been developing and using integrated software for personal computer systems for the past 8 years. The software encompasses data collection, data transfer, data base management, hydrologic modelling (storms, stormwater, water quality, hydraulics, receiving waters) and real-time control. Most of the programs are written in FORTRAN.

This paper describes the relationship between these activities and the sequence of data processing necessary for various types of study. About 50 programs are listed and briefly described - most are geared to the acquisition of continuous high-resolution rainfall data, subsequent data base management, continuous stormwater modelling and real-time control.

Performance of the programs in an IBM-PC compatible hardware environment is indicated in broad terms; integration of this range of hydrologic software is now being completed.

INTRODUCTION

Long-term flow and pollutant records may be synthetically generated using an elaborate, deterministic computer model such as the U.S. Environmental Protection Agency's Stormwater Management Model (SWMM)(1). Version 3 of SWMM has been adapted by our group to run on IBM-PC compatible machines, so that a complete, very large and intricate model can now be run on a computer costing less than $1500 US (2). There are no serious drawbacks using PCSWMM3 in this environment. We have supplied this package to a significant number of users, and the existing large SWMM user group infrastructure assures wide-ranging, continuing support and interest in this product. The package is being used world-wide.

PCSWMM3 has many advanced capabilities not available in other models in use today, e.g. menu-driven design dialogue, input error checking, diagnostic messages, transparent file handling, verification/validation

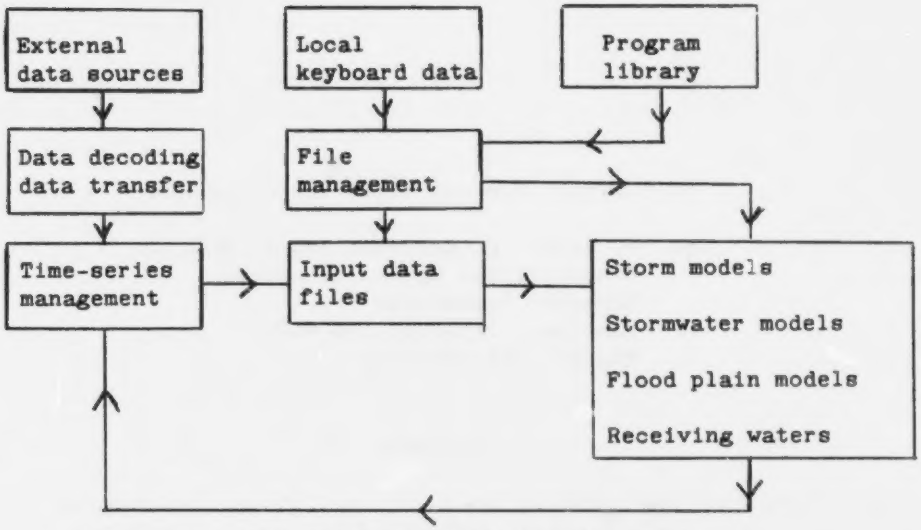

Figure 1. Hydrologic computing environment.

modules, continuous modelling, water quality, diversion structures, sedimentation and scour, costing, screen-oriented graphics, statistical summaries, to name a few. Nevertheless the package is easy to use, especially because of the user-friendly documentation, simple menu-driven input and graphical output. Probably the most important attribute is continuous modelling, since this removes all the intrinsic shortcomings of the design storm approach - all flows are ranked directly, giving correct probabilities and risks (3). Difficulties due to the management of large amounts of input and output data are eliminated by our special data base management software (CHGTSM), also specially written for IBM-PC compatible environments (4). The general computing environment is shown in Figure 1.

One question that is seldom meticulously addressed is model validation. We have devised a methodology incorporating systematic verification, sensitivity analyses, calibration and validation, that appears to work smoothly and faultlessly (5). The procedure is set out in the PCSWMM3 Workbook Module. Of course, adequate field data is essential, and our inexpensive raingauges and data loggers (6) render it easy to collect sufficient summer thunderstorm activity in one or two months (in the north during the period May-October) to satisfy model calibration for most critical modelling situations. The same instrumentation is used in our real-time controllers (7). This points to the advantages of considering an integrated microcomputer-compatible system from the outset. A typical sequence of activities described here is shown schematically in Figure 2.

There are other important benefits that accrue from a dense network of field stations. Perhaps the best example occurs in storm modelling. It should be obvious that urban flooding often results from the short, sharp, local convective storms that are so common in warm weather. These storms are much smaller in area than the catchments and have surprisingly short lives, typically 30 minutes or so. Our software PCRAINPAC analyses data from a network of synchronized raingauges to compute storm speeds, directions, growth and decay (8). Output from PCRAINPAC is fed directly into PCSWMM3 (9) to compute flood flows and pollution loads that are demonstrably more accurate than any other hydrologic package known to us. This is to be expected: the better the input, the better the results.

Figure 2 presents the typical sequence of activities involved in carrying out a hydrologic modelling study on a personal computer environment. Any such study requires time series' of rainfall as input to the model. A record of discharge is required in order to calibrate and validate the model. These data are typically supplied by government agencies such as the Atmospheric Environment Service (AES) and Water Survey of Canada (WSC) in Canada and the National Weather Service (NWS) in the United States.

Large amounts of data are often required, especially for studies involving continuous simulation. This data can cover periods in excess of 40 years. It would not be feasible to distribute or acquire such data on media such as floppy diskette or over telephone lines. Consequently, the most efficient method for supplying/acquiring this data is on 9-track magnetic tape. Several commercially available subsystems exist which permit interfacing of standard, 9-track, mainframe-compatible tape drives with IBM-PC compatible microcomputers. These subsystems interface through a

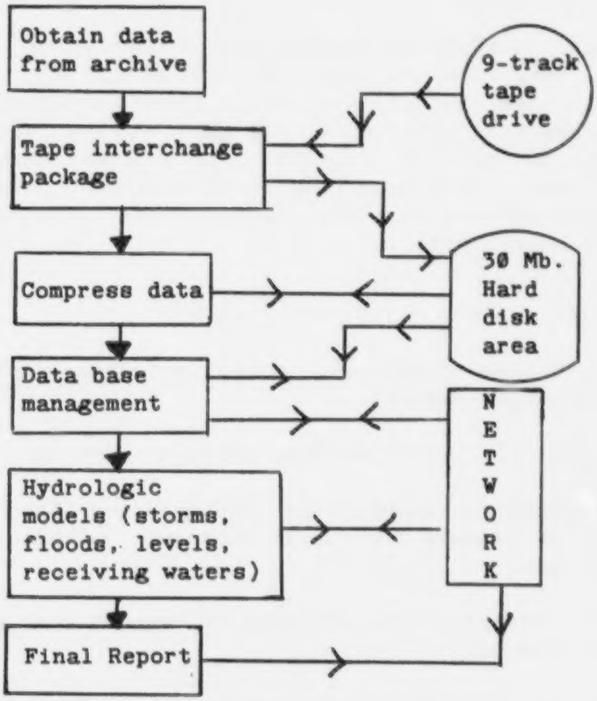
Obtain data from archive
9-track tape drive
Tape interchange package
Compress data
30 Mb. Hard disk area
Data base management
NETWORK
Hydrologic models (storms, floods, levels, receiving waters)
Final Report

Figure 2. Typical sequence of activities.

board installed in one of the expansion slots on the PC bus. The tape interchange utility supplied with the subsystem downloads the data to the user's system.

Depending on the amount of data required for the study, the files created from the data supplied on tape can range from 1 to 60 Mbyte in size. This necessitates having sufficient storage available on the users system. It has been our experience that a hard disk drive with 40 to 50 Mbyte capacity can be acquired fairly inexpensively and will satisfy the needs of most studies requiring large data storage.

In order to properly manage, retrieve and update this database of hydrometeorologic data special software is required. The software which has been developed has the capability to take the data in its raw form and store it in a "compressed" form in which only non-zero data is retained thereby reducing the data file size. In an application performed by our group, a rainfall database covering the period 1975-1984 with an uncompressed size of 52 Mbyte was reduced to 18 Mbyte using the data compression technique. The database, once constructed, is coherent and can be made available to all members of a modelling group. Updating or correction of the database is the responsibility of one person who needs only manipulate a single file. This ensures that all modellers are using an identical database. Increased study reliability results. The data is accessed by users through menu-driven retrieval utilities and can be automatically interfaced to simulation package such as PCSWMM.

Such a hydrologic database management system when being used by a group of modellers has been found to function most efficiently in a network environment. The database and application programs reside on a central network computer, or server. The modelling group uses a number of PC's which are also nodes on the network. Applications programs run locally and independently on each node access data from the simulation. Results can be returned to the database for use by other applications. The applications programs are described subsequently.

Flood and pollution control may be effected by:

a) diverting overflows to storage, directly to receiving waters or to some other sewer network,

b) pumping or draining overflows from detention storage,

c) warning responsible authorities of impending flooding and/or pollution, for example for timely closure of swimming beaches or underpasses, and

d) computing the extent of the swimming or recreational areas which should be evacuated.

Real-time control of urban stormwater may be achieved by onsite microcomputers rather than by a single, central computer. The microcomputers need not be expensive; in fact, $30 hand-held Z-80 based microcomputers having BASIC in ROM are probably sufficient at each control site (7). The site will generally include:

1) an instrument to measure flow, water level, rainfall and/or pollutant concentration,

2) circuitry to transform the signal into information meaningful to the microcomputer,

3) a small, replaceable microcomputer running a model derived from statistical or similar analysis of data relevant to the control site,

4) circuitry to drive the electrical control mechanism, and

5) a data logger recording all incoming information, the control action, and the precise date and time.

The microcomputers and associated circuitry, for onsite control, and inexpensive raingauges measuring rainfall intensity, require large amounts of software, also developed by the Computational Hydraulics Group at McMaster University. The real-time control software (TSDASUTIL) has been based on transfer-function models (TFM) of synthetic long-term flow and pollution at each control site. The TFM is easily coded into the data logging program, written in BASIC. TSDASUTIL is menu-driven and has, among other attributes, data communication software, so that the data may be automatically transferred to a central database (10) after manually retrieving the tape. Other software that may typically be involved in such a study is depicted in Figure 3.

The TFM software is derived directly from the synthetic long-term time series generated by continuous PCSWMM3 at each site. It predicts expected flow and pollution in the vicinity of the control site a few minutes ahead (enough to complete a control action). There is a risk, in some drainage systems, that bad timing of diversions could cause flows to coincide downstream such that flow conditions become worse than they need have been. In these cases it is necessary to run the continuous simulation for the overall drainage system with all local control software at each site simulated. This is why an elaborate overall deterministic model is necessary. PCSWMM3 also allows the full range of other stormwater management strategies (separation of roof leaders, tile drains, etc.) to be evaluated (10).

The minimum functions that must be supported in a real-time flood control system are:

a) data acquisition,
b) data storage/retrieval,
c) streamflow forecasting,
d) project operation.

The software which might be utilized a typical RTC application is illustrated in Figure 3. These programs perform the four tasks listed above.

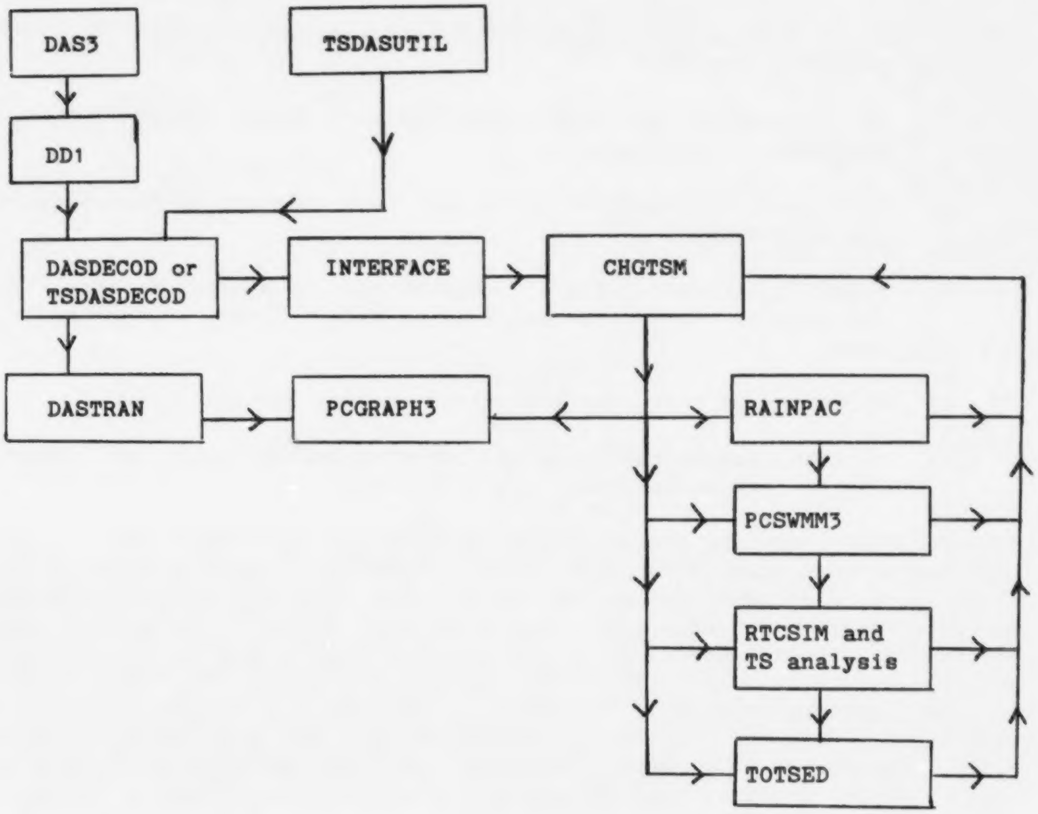

Figure 3. Typical Application (evaluation of RTC).

DAS3 and TSDASUTIL collect data from nearby raingauges and store it on cassette tapes in the field. TSDASUTIL might also be controlling a diversion structure, locally, based on the incoming real-time rainfall data. After collecting the cassette tapes and transporting them to a central site, the data is decoded and communicated to our local area network (LAN) of IBM-PC compatibles. The LAN consists of a 52 Mbyte hard disk, a 9-track magnetic tape drive, a line printer, and three microcomputers, each with 512 kbytes of RAM, 8087 co-processors, and two floppy disk drives. Program DD1 decodes the data collected by DAS3, using a data decoder. TSDASUTIL transmits the data via a RS232 interface. Data is received by DASDECOD or TSDASDECOD through the serial port of one of the LAN stations, and stored in a disk file.

The data can be processed immediately by DASTRAN or permanently archived in the CHGTSM for a variety of applications. DASTRAN translates the raw data file created by DASDECOD or TSDASDECOD into event summaries. If desired the program also produces two additional files: 1) a PCSWMM3 compatible interface file for input to the PCSTATISTICS3 module, useful for ranking rainfall events, based on peak intensity, total volume, duration, etc., and 2) a PCGRAPH compatible data file for plotting hyetographs. INTERFACE reformats the raw data for entry to the CHGTSM, our rainfall-runoff data base, which resides on the LAN. This makes the data available to our group for a number of applications, including data processing, graphics, report generation, and hydrologic modelling studies.

Figure 3, presents a typical hydrologic modelling application, aimed at evaluating proposed RTC schemes. First RAINPAC is used to prepare rainfall input for the PCSWMM3 continuous model. STOVEL is used to analyse a number of point rainfall records for direction, speed, and peak growth/decay mechanisms. THOR4DPT uses the results of the STOVEL analysis to generate hyetographs for each subcatchment in the PCRUNOFF3 model. The continuous water quantity and quality record generated by PCRUNOFF3 is then input to the PCTRANSPORT3 model of the diversion structure(s). The results of the continuous PCRUNOFF3 and PCTRANSPORT3 simulations can be used to develop a simple rainfall-runoff forecast model which can be evaluated in RTCSIM, and implemented in TSDASUTIL. TOTSED models the deposition and scour of sediment at a combined sewer outfall.

Other programs which might be included in such an analysis are PCSTORAGE3, CHGQUAL, OVRFLO3, and TOTSED. At any point in the hydrologic modelling, the rainfall, runoff, and pollutant time series output by these programs can be stored in the CHGTSM for subsequent analysis.

The whole system may sound unduly complex, but runs very effectively on IBM-PC compatibles. All of the programs are operational; the software is listed below.

1. STORMWATER MANAGEMENT

The PCSWMM3 package for urban runoff quantity and quality in storm and combined sewer systems comprises 11 modules:

1. PCRUNOFF3 module - simulates overland flow hydrographs and pollutographs from urban and non-urban watersheds.

2. PCTRANSPORT3 module - generates infiltration, dry weather flow and provides hydraulic routing model for flow and pollutants in non-surcharged situations. Includes pipe design options and various pre-programmed pipe shapes and regulators.

3. PCEXTRAN3 module - hydraulic routing model for flow in surcharged situations. Includes regulators, pump stations, tides and some pre-programmed pipe shapes.

4. PCSTORAGE3 module - simulates the flow routing and removal processes of detention ponds and sewage treatment plants. Includes cost estimation procedures.

5. PCSTATISTICS3 module - provides simple statistical analysis and frequency analysis of continuous simulation results generated by other modules.

6. PCGRAPHICS3 module - provides a general plotting utility for PCSWMM3 results and other data.

7. PCEXECUTIVE3 module - assists the user in data preparation for PCSWMM, controls file handling.

8. PCCOMBINE3 module - provides a utility for merging files of results from individual PCSWMM3 simulations for input to subsequent modules thereby allowing large areas to be simulated in segments.

9. PCWORKBOOK3 module - leads novice user systematically through a series of verification tests on several modules of PCSWMM3.

10. PCINSTAL3 module - provides an "official" check on the performance of the users hardware and software system.

11. PCSWMM3 - RUNOFFG - a modified version of the RUNOFF module which includes interflow simulation. This provides a means for continuous computing of startup values for wet weather events.

12. CHGQUAL - modified version of the SWMM3 RUNOFF block water quality modelling algorithms, for pollutant buildup and washoff.

13. OVRFLO3 - given the geometry of the main channel and side weir, computes the water surface profile along a side weir, and the over-spill, given incoming flow.

14. TOTSED - analyses quasi-transient sediment motion, deposition and scour at a combined sewer outfall.

Expert System:

15. PCSWMM A-S (Auto Sensitivity) represents a group of program modules forming a shell around the PCSWMM program. The program modules work on an interactive basis with the user and the RUNOFF module of PCSWMM. They provide sensitivity and uncertainty information on the effects of eleven physical input parameters required by the RUNOFF module. The output generated through PCSWMM A-S includes a ranking of sensitivity magnitudes for each of the tested parameters and the uncertainty, expressed as a percentage, that is associated with the output from the RUNOFF module.

    The program requires the standard minimum hardware configuration specified for the PCSWMM3 package. This includes an IBM-PC or compatible with at least 512k RAM, two 360k drives and an 8087 NDP. To operate the program the user responds to a series of prompts contained in the batch "shell" and indicates an existing file which is to be analysed by the A-S. Input required includes confidence levels in the input parameters and certain utility data. Analysed output data includes information on total volume of runoff, peak flow, mean flow, time to peak flow, flows exceeding prespecified limits and the volume of any exceedance. Output is directed to the console as well as to files on the disk drive. The time required for a complete run of the package is a minimum of 60 to 80 minutes for a single subcatchment with 12 data points for the input hydrograph. Internal processing of the data requires approximately 80% of the total run time.

2. STORM MODELLING

RAINPAC is a package of 4 programs which analyses and models the kinematic properties of urban rainfall events:

16. STOVEL - analyses a number of point rainfall records for direction, speed and peak growth/decay mechanisms

17. THOR4DPT - simulates the kinematic properties of a storm using STOVEL results to generate hyetographs at point locations.

18. THOR4D - simulates kinematic and areal averages for storm events.

19. THOR3D - simulates the areal average of a line storm event.

3. FLOOD PLAIN ANALYSIS

20. PCHYMO - hydrologic modelling language for simulating non-urban watersheds on a simple event basis using the SCS curve number technique. Methodology not recommended for serious hydrologists.

21. HEC-2 - a steady state backwater curve calculation model which simulates water surface elevations along open channels.

22. FASTHEC-2 - a preprocessing program for HEC-2 which assists in data preparation.

4. TIME SERIES MANAGEMENT SYSTEM (TSM)

The following 5 programs are used to build and/or update the data base:

23. OPENFL - opens a TSS file and optimizes record size and number of records in a file.

24. OPELBL - opens a data set in the file opened by OPENFL.

25. PATCH - reads the contents of a data set opened by OPELBL (created by user previously) and inserts it into the data base.

26. INSERT - similar to PATCH (replaces current data with new data).

27. UPDATE - similar to PATCH.

The following 5 programs are used to retrieve data. QUERY and CONTINS are similar to RETRIV, and could be classed as output handlers for specific purposes.

28. RETYDIR - displays information regarding the types of data which reside in the data base, and selects the appropriate TSS file, containing the data requested by the user.

29. RETSSDIR - displays information regarding the locations for which data reside in the directory.

30. RETRIV - retrieves data for user specified period (1-365 days) in constant or variable timestep format.

31. QUERY - retrieves and displays data for user specified site and period. A disk file containing this data, at either constant or variable timesteps may be created.

32. CONTINS - similar to QUERY, but the user may specify the format of the output file, and thus can be used to prepare input data files for other programs. Default format is for PCSWMM3 RUNOFF data file (Rainfall data groups).

33. INTERFACE - utility for processing data (flow, rainfall) from government agencies such as National Weather Service (US), Atmospheric Environment Service (Canada), Water Survey of Canada, or private groups such as our group (CHG), into forms suitable for input to PCSWMM3 or the CHG-TSM.

4. DATA MANAGEMENT

34. INTERP - processes rainfall from decoders or flow from chart records into event summaries. Also provides input to FASTPLOT. Similar version for Apple IIe, written in Applesoft BASIC. Also FORTRAN version for APIOS DCPS-D/WPS units.

35. DASTRAN - processes raw rainfall data as created by DASDECOD or TSDASDECOD into event summaries (similar to INTERP). Also creates PCSWMM3 compatible interface file of rainfall, and PCGRAPH input file for plotting data.

36. TSDASUTIL - program package performs various tasks related to data acquisition and RTC, including collecting rainfall data and controlling a simple diversion structure in real-time. The package also processes the data, plots hyetographs, and transmits the data to IBM-PC compatibles for further processing; written in Sinclair BASIC for the Timex/Sinclair 1000 microcomputer. Two main modules handle these tasks: 1) RTCONTROL - data acquisition and RTC, and 2) TSDECODER - data processing, plotting, decoding, and communication.

6. DAS PROGRAMS (DAS = Data Acquisition System)

37. LPDAS2.V2 - microprocessor controller program for timing and data collection for DCPS and low-power data logger; written in macro-assembler.

38. DAS3 - same as above for standard data logger.

39. TBDAS3 - same as above but for TBRG.

40. DASMOE1 - same as DAS3 but also records data from wet/dry precipitation sampler.

7. DAS Data Decoder Programs

41. DD1 - decodes data collected by DAS on cassette tape, and transmits it to computer (IBM-PC compatibles, TRS-80, PDP 11/23); written in macro assembler.

42. DD1-APP - same as above but for Apple IIe.

43. TSDECODER - retrieves data from storage, converts from Sinclair to ASCII character set, and transmits data serially to IBM-PC compatibles. Also performs some simple processing of the data in the field, including simple graphics; part of TSDASUTIL package (see 36).

44. DASDECOD - receives data sent by TSDASUTIL through RS232 device and creates a disk file (same as DASDECOD) on an IBM-PC compatible; written in GWBASIC.

45. TSDASDECOD - receives data sent by TSDECODER module of TSDASUTIL through RS232 device and creates a disk file (same as DASDECOD) on an IBM-PC compatible; written in GWBASIC.

46. DASINTERFACE - receives data sent by decoder (DD1-APP), creating a disk file on Apple IIe; written in Applesoft BASIC.

8. PLOTTING PROGRAMS

47. FASTPLOT - plots hyetographs or hydrographs of events, input from INTERP program; for use with Houston Instruments plotter.

48. PCGRAPH - (same as GRAPHICS module of PCSWMM3), used to plot hyetographs as created by DASTRAN; written for IBM-PC compatibles.

8. REAL-TIME CONTROL

49. RTCONTROL - see 36.

50. BOXJEN (MITS originally) - program for the identification of a discrete linear Transfer Function Model from input and output time series; written by the Chemical Engineering Process Control Group at McMaster University modified for use on IBM-PC compatibles.

51. RTCSIM - simulates the operation of a simple diversion structure, continuously, in real-time. Uses the results of a TRANSPORT simulation, and runoff forecasts to simulate the diversion of CSO.

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DABRO
A BASIC LANGUAGE PROGRAM FOR HYDROGRAPH COMPUTATION

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ABSTRACT

In this paper a hydrologic model designated DABRO for the computation of hydrographs from complex drainage basins is presented and discussed. In this model, written in the BASIC language, rainfall-excess increments, computed by the SCS runoff curve number procedure, are applied to a mathematically computed unit hydrograph of each subbasin to obtain the runoff design hydrograph from each subbasin. These subbasin hydrographs are then summed and/or routed downstream by the model to obtain the total basin hydrograph. The hydrographs computed by the model are similar to those computed by TR-20 and HEC-1. However, because of easy data entry and editing capabilities, the model is extremely easy to use. Also, the code may be easily changed by the user to fit particular situations.

Simulations of actual runoff events from three urban watersheds and one large rural watershed by use of this model are also presented and the results discussed.

INTRODUCTION

DABRO, in its "batch model format", actually consists of three separate programs or parts. The first program (Part 1 - Data Input Files) is used to input the various subbasin properties (subbasin areas, runoff curve numbers, unit hydrograph and routing parameters, etc.) on to a disk file; the second program (Part 2 - Read Data Files) is used for reading and/or editing the data placed in disk file by the first program; and the third program (Part 3 - Hydrograph Computation) computes the rainfall-excess, unit hydrograph and total hydrograph of each subbasin, and sums and/or routes them downstream to the basin outlet. In this version of the model, the total hydrograph of each branch is computed in a sequential manner and stored on disk file. Later, during subsequent program operation as the main channel hydrographs are being computed, these hydrographs are read from disk file by the program in the same order in which they were input. The program then adds the branch hydrographs to the main channel hydrographs to produce the total hydrographs below the channel intersections. A definitive numbering sequence is necessary to insure proper program operation.

A separate program (RAIN.FIL) is used to input rainfall increments comprising the design storm on disk file.

These programs, plus programs to perform the various operations separately (compute hydrographs, sum hydrographs, flood route hydrographs), constitute the book "Design Hydrographs by the Drainage Basin Runoff Model (DABRO), published by Hilbern Engineering Software.

In the model, rainfall-excess increments are computed from successive rainfall increments by a modification of the standard Soil Conservation Service (SCS) rainfall-runoff equation which modification accounts for runoff from three sources - from the urban directly connected impervious area; from the urban pervious (grassed) and nondirectly connected impervious area; and from the rural portion of each subbasin or from each source only. The mathematically computed synthetic unit hydrograph used in the computation of the design hydrograph of each subbasin, although similar in shape to the unit hydrograph of F. F. Snyder, permits the user to vary both its peak flow value (discharge) and the time to peak (lag time) of the peak flow. Reservoir routing is accomplished by the storage-indication working curve method and channel routing by the Muskingum Method.

In addition to successive rainfall increments comprising the design storm, inputs to the various programs are the number (designation) of each subbasin; the properties of each subbasin including:

1. the previously computed peak flow (discharge) and time to peak of the unit hydrograph of the subbasin,
2. the area of the subbasin,
3. the percentages of the directly connected impervious area and rural areas of the subbasin,
4. the runoff curve number (CN) for the urban pervious and non-directly connected impervious area of the subbasin,
5. the runoff curve number (CN) for the rural portion of the subbasin,
6. the initial abstraction (IA) for the urban pervious and non-directly connected and rural portions of the subbasin;

the properties of the various channel reaches through which flood routing is being accomplished, including:

1. the length of the river reach (reach length) through which the subbasin hydrograph or combined subbasin hydrographs are to be routed,
2. the reach velocity,
3. the reach routing coefficient (X);

the properties of the storage reservoir through which flood routing is to be accomplished, including:

1. discharge rates from the reservoir at various elevations above the spillway,
2. the storage volume in the reservoir at these same elevations;

and the computational time interval.

Outputs from the programs are a table of the original successive rainfall increments input to the programs and the resultant computed rainfall-excess increments; a table of the computed unit hydrograph ordinates of each subbasin; tables of the total combined (summed and/or routed) hydrographs of the various subbasins; and a plot of the final total basin hydrograph. Included with the tables of hydrograph ordinates (unit, design, summed and routed) are the number of inches of rainfall-excess from each subbasin (=1.0±" for the unit hydrograph).

RAINFALL-EXCESS

In the DABRO Model, rainfall-excess increments are computed by a modification of the standard rainfall-runoff equation of the Soil Conservation Service.(1) In this modification, as previously stated, direct runoff from a drainage basin is considered to come from three or less sources - from the urban directly connect impervious area; from the urban pervious (grassed) and non-directly connected impervious (urban) areas and from the rural area of the basin, each of which is considered separately.

In the original SCS procedure, a runoff curve number CN for a particular entire subbasin was first computed which enabled the computation of the direct runoff Q up to a certain time from the basin knowing the rainfall P which fell on the basin up to that time. In the originally derived procedure, the following equations were used to compute direct runoff Q from rainfall:

$$Q = \frac{(P-IA)^2}{P-IA+S} \tag{1}$$

$$CN = \frac{(1000)}{10+S} \tag{2}$$

where

Q = Direct runoff (inches)
P = Total storm rainfall (inches)
IA = Initial abstraction (inches)
S = Watershed storage factor (inches)
CN = Runoff Curve Number (dimensionless parameter)

The runoff ΔQ in inches which occurred in a certain time interval Δ_t was then equal to (Q_t-Q_{t-1}).

The Runoff Curve Numbers for various types of agricultural, suburban and urban land uses as originally developed by the SCS for computing direct runoff from rainfall utilizing these equations are given in various SCS manuals. As most engineers know, these Runoff Curve Numbers depend on the

Hydrologic Soil Group, the land use or cover and the Antecedent Moisture Condition for the particular drainage basin.

For purposes of computing direct runoff by the method presented in this program, the following equation is used:

$$\Delta Q_t = \frac{\Delta P(\%DCIA)}{100} + \Delta Q_u \frac{(100-\%DCIA-\%RURAL)}{100} + \Delta Q_r \frac{(\%RURAL)}{100} \quad (3)$$

where ΔQ_t is the total direct runoff from the basin in a particular time interval Δt $(=Q_t-Q_{t-1})$; $\frac{\Delta P(\%DCIA)}{100}$ is the direct runoff from the directly connected impervious area (DCIA); $\Delta Q_u \frac{(100-\%DCIA-\%RURAL)}{100}$ is the direct runoff from the pervious areas and non-directly connected impervious (urban) areas in this time interval as computed by Equation 1, $\Delta Q_r \frac{(\%RURAL)}{100}$ is the direct runoff from the undeveloped (rural) areas in this time interval as also computed by Equation 1, and ΔP is the rainfall that falls on the basin during this time interval. The ability to compute runoff from three separate type areas within a subbasin enables the evaluation of land use changes occurring on the basin.

In the computation of direct runoff from the directly connected impervious area (DCIA) by this program, the first 0.1 inch falling on such areas is assumed to fill up depression storage and, therefore, is not assumed to contribute any runoff.

In the original procedure as developed by the SCS for use in rural areas, an Initial Abstraction of 0.20S was used in Equation 1 to complete direct runoff. However, in the application of the procedure as modified herein the following values of IA can be used as being more representative of the initial abstractions imposed on runoff by urban areas(2):

Hydrologic Soil Group	IA
A	0.075S
B	0.10S
C	0.15S
D	0.20S (unchanged)

Where runoff from undeveloped (rural) land is computed, the value of 0.20S (unchanged) should be used.

In the DABRO Model, runoff can be computed from completely urbanized basins in which case zero values are input to the program for the Runoff Curve No.-Rural; the Initial Abs. Coef.-Rural and for the Percent-Rural. If the basin is completely rural (no urbanization), zero values are input to the program for the Runoff Curve No.-Urban Pervious & DCIA; the Initial Abs. Coef.-Urban Pervious & DCIA; and the Percent-DCIA.

In the computation of runoff by the SCS procedure and as modified herein, three Antecedent Moisture Conditions (AMC) are defined depending upon the amount of rainfall (antecedent rainfall) that has fallen on the basin prior to the storm from which direct runoff is being computed, which AMC's are defined in Section 4, Hydrology of the SCS's National Engineering Handbook.

In the computation of a Runoff Curve Number (CN) for a particular basin or portion thereof, by the SCS procedure, a weighting process, depending on the percentage of the particular land use cover, is used. As previously mentioned, separate Runoff Curve Numbers for both the urban portion of the Drainage Basin not directly connected - the pervious (grassed) area and the non-directly connected impervious area of the urban portion and the rural portion of the subbasins are both program inputs.

Since the DABRO Model uses the SCS Runoff Curve Number to compute rainfall-excess, it is, of course, a single event model. However, as subsequently discussed, it has been and can be easily modified for continuous analysis using an antecedent precipitation index (API) which relates rainfall to S, the watershed storage factor, or CN, the Runoff Curve Number.

UNIT HYDROGRAPH

In the DABRO Model, the ordinates of a synthetic unit hydrograph for each basin are computed by the following equation developed by James A.

Constant, formerly Chief Reservoir Regulation Unit, Albuquerque District, Corps of Engineers(3).

$$Q = \frac{111.8094\ A\ e^{-\frac{\left(\text{Log}\ \frac{t}{T}\right)^2}{2\sigma^2}}}{\sigma t} \qquad (4)$$

where

Q = Flow Rate (Discharge) in cfs
A = Drainage basin area in square miles
t = Time in hours from beginning of runoff
T = Time in hours at which 50% of flow has passed
  $= 10\ (\text{In}\ 10\sigma^2 + \text{Log TmaxQ})$
σ = Standard deviation of the log 10 of the values (see note below)

This equation produces hydrographs that generally conform to the shape of the synthetic unit hydrograph developed by F. F. Snyder as presented in Corps of Engineers Manual EM-1110-2-1405 - the rising limbs of the unit hydrograph are parabolic and the recession curves are approximately exponential and are similar to the unit hydrograph of the Soil Conservation Service.

In the evaluation of Equation 4, Q is differentiated with respect to time and set equal to zero - standard procedure for determining the peak value of a function - which results in the following equation (after transposing and rearranging):

$$\frac{e^{-\frac{(\text{In}\ 10)^2}{2}\sigma^2}}{\sigma} = \frac{\text{TmaxQ}\ \ \text{Qmax}}{111.8094A} \qquad \text{(See note below)} \qquad (5)$$

Note: σ = Greek letter SIGMA; letter S used for SIGMA in program listing.

$\frac{(\text{In}\ 10)^2}{2}\sigma^2 = 2.650949056\ \sigma^2$

The values on the right side of Equation 5 are generally easily computed from standard equations as will be subsequently described. TmaxQ is the time (in hours) from the beginning of rainfall-excess to the peak of the unit hydrograph (TOTAL TIME TO PEAK-HOURS) which is equal to the lag time (normally defined as the time from center of mass of rainfall-excess to the time of peak) plus one-half of the unit hydrograph duration or tp + D/2.

In this program for computing unit hydrographs, σ is first computed by use of Equation 5 using Newton's Method of Successive Approximations. The time and discharge at the end of each time interval are then computed by Equation 4.

The maximum discharge and time to peak of the unit hydrograph can be computed by the standard SCS equations:

$$Tp = \frac{D}{2} + L = \frac{D}{2} + 0.6Tc$$

and

$$qp = \frac{K\ A\ Q\ (=1")}{Tp}$$

where

qp = Unit hydrograph peak flow (discharge) in cfs
Tp = Total time to peak in hours
D = Duration of unit hydrograph in hours (must be equal to time increment Δt)
A = Drainage basin area in square miles
L = Basin lag time in hours (=tp)
Tc = Time of concentration in hours
K = Unit hydrograph peak flow factor (=250 to 600, normally 484),

by the Snyder equations; or any other accepted method.

SUBBASIN AND BASIN HYDROGRAPHS

In the DABRO model, the ordinates of the design hydrograph of each subbasin are computed by multiplying the successive ordinates of the computed unit hydrograph by the computed successive rainfall-excess increments and then summing up the values so obtained which sum is the ordinate of the design hydrograph. The program automatically performs the required time increment lagging required for hydrograph computation by this procedure. Flood routing through channel storage is accomplished by the Muskingum Method and flood routing through reservoir storage by the storage-indication working curve method or Modified Puls Method.

In the model, the total basin (outflow) hydrograph is computed by summing up the various subbasin design (and/or routed) hydrographs. The model automatically performs the time lagging required for basin hydrograph computation. As previously stated, the DABRO Model will also plot up the final outflow hydrograph from the total basin under consideration (time in hours vs discharge in cfs) if desired. The final basin outflow hydrograph can also be saved on disk for possible further manipulation by the other programs in the book.

MODEL VARIATIONS

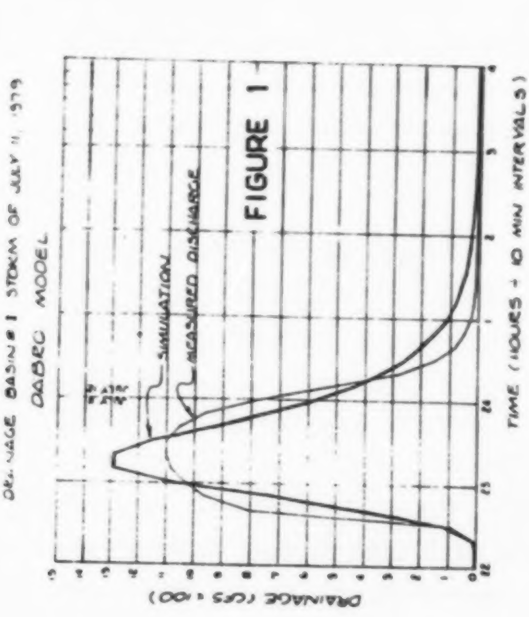
Because the DABRO Model program is written in Microsoft Basic and the program can be listed (not copy protected), changes can easily be made. For instance, data can be easily imbedded into the program so that manual entry of subbasin variables does not have to be input to the model every time it is used. Also, modifications to allow manual entry of a series of rainfall increments so that a different set of rainfall increments can be applied to each subbasin can easily be done. As subsequently described, this enables the model to be used in real-time reservoir regulation.

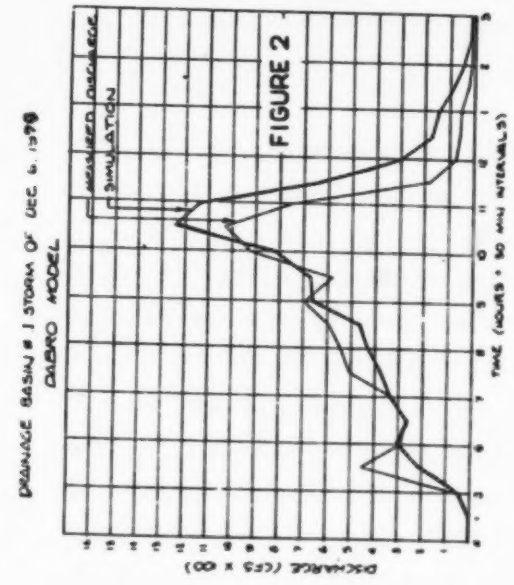
MODEL APPLICATION

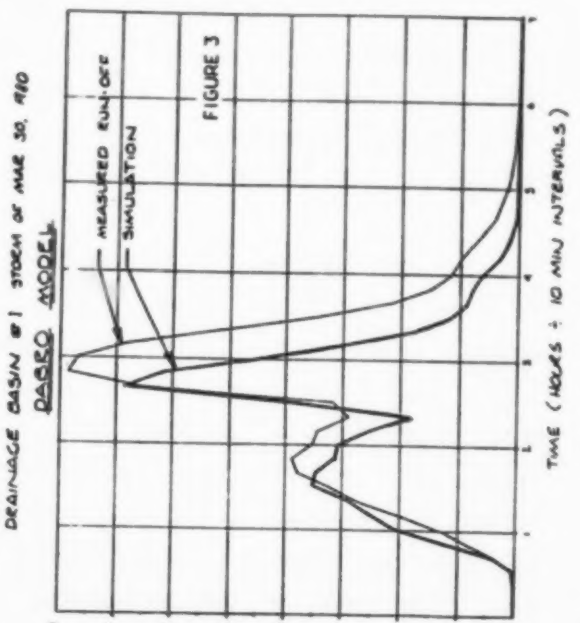
General: The DABRO Model has, to date, been used to simulate runoff from four different basins. Three of the basins were small, fairly steep, almost completely urban basins located in the Tallahassee area (hereinafter referred to as Basin Nos. 1, 2 and 3). The fourth basin (Basin No. 4) was a large, fairly flat, sandy, almost completely rural basin, located in Manatee County, Florida (the Manatee River basin). All were well documented as to their area, physical properties, land use, soil type, etc., and all had accurate gaged rainfall-runoff data including information on the amount of antecedent rainfall prior to the event simulated. In all cases, the Theissen polygon method was used to weigh the rainfall increments of the various storms simulated.

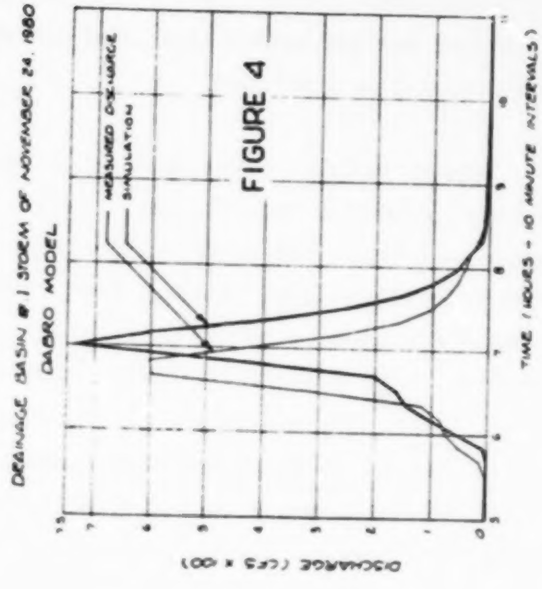
Urban Basins: Basin Nos. 1, 2 and 3 are, as previously stated, all highly urbanized basins, were gaged as part of the USGS's urban basin gaging program, conducted in the Leon County, Florida area. Basin No. 1, the Frenchtown Creek basin, 3.29 square miles in size, is essentially a completely sewered basin consisting of the Frenchtown Creek area and the Florida State University (FSU) campus. The lower portion of the Frenchtown Creek basin consists of a 1200 feet long, 8 feet diameter circular conduit which carries Frenchtown Creek under the FSU campus. Basin No. 2, the Franklin Avenue drain basin, 2.06 square miles in size, is also essentially a highly urbanized basin. It consists of two definitive portions or parts; the upper one-half, which consists of a sewered residential area, and the lower one-half, which consists of a portion of the downtown area of Tallahassee and the State Capitol complex. The open channel draining this lower portion of the basin is essentially retangular or trapezoidal in shape, much of which channel is lined. Basin No. 3, 0.21 square miles in size, is an essentially mixed residential and commercial area drained by a fairly steep open channel. Soils in all three basins were classified by the USGS as being in Hydrologic Soil Group B.

In all, sixteen rainfall events were simulated by the DABRO Model; five on Basin No. 1, five on Basin No. 2 and six on Basin No. 3. The results of these simulations are shown on Figures 1 through 16. In all cases, the total impervious area and directly connected impervious area were determined from a study of aerial photographs supplemented by field inspection. Runoff Curve Numbers for the pervious and non-directly connected impervious areas were determined in the usual manner by weighing the grass and impervious areas. In all cases, the time to peak in hours and peak flow rate of the ten-minute unit hydrograph of each basin was determined by the equations developed by Espey, Altman and Graves(4). In accordance with program requirements that the time step Δt be equal to the

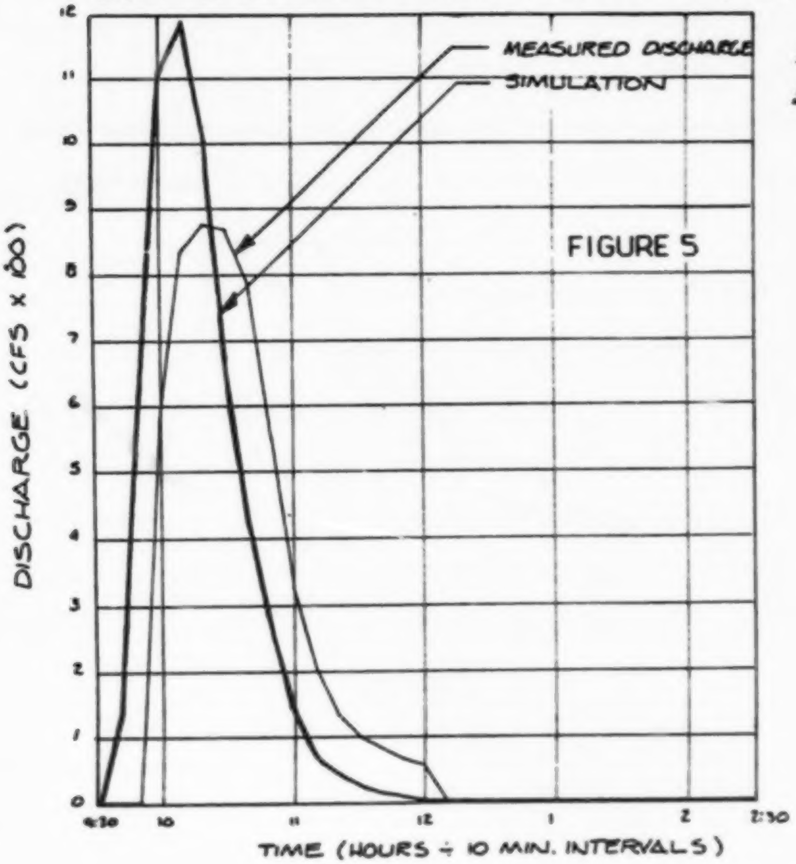

DRAINAGE BASIN #1 STORM OF JULY 11, 1979
DABRO MODEL
SIMULATION
MEASURED DISCHARGE
FIGURE 1
DRAINAGE (CFS ÷ 100)
TIME (HOURS + 10 MIN INTERVALS)


DRAINAGE BASIN #1 STORM OF DEC 6, 1978
DABRO MODEL
MEASURED DISCHARGE
SIMULATION
FIGURE 2
DISCHARGE (CFS x 100)
TIME (HOURS + 30 MIN INTERVALS)


DRAINAGE BASIN #1 STORM OF MAR 30, 1980
DABRO MODEL
MEASURED RUN-OFF
SIMULATION
FIGURE 3
DISCHARGE (CFS)
TIME (HOURS ÷ 10 MIN INTERVALS)


DRAINAGE BASIN #1 STORM OF NOVEMBER 24, 1980
DABRO MODEL
MEASURED DISCHARGE
SIMULATION
FIGURE 4
DISCHARGE (CFS x 100)
TIME (HOURS - 10 MINUTE INTERVALS)

DRAINAGE BASIN No. 1 STORM OF MARCH 30, 1981



DABRO MODEL

FIGURE 5

DRAINAGE BASIN # 2 STORM OF JULY 11, 1979

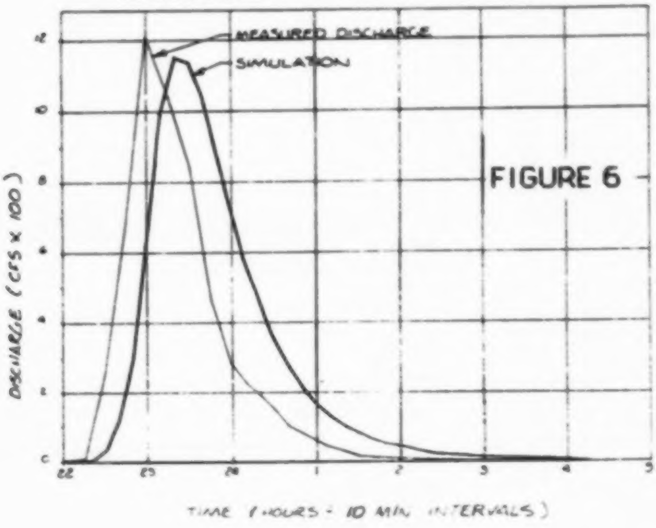


FIGURE 6

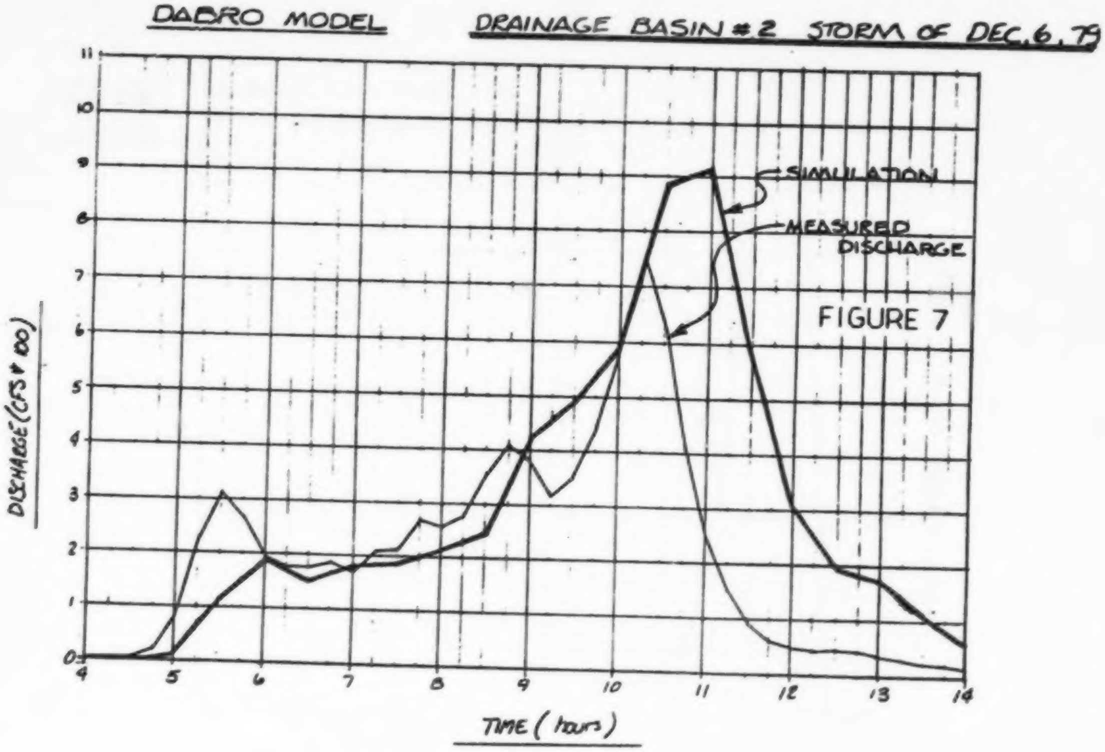


FIGURE 7

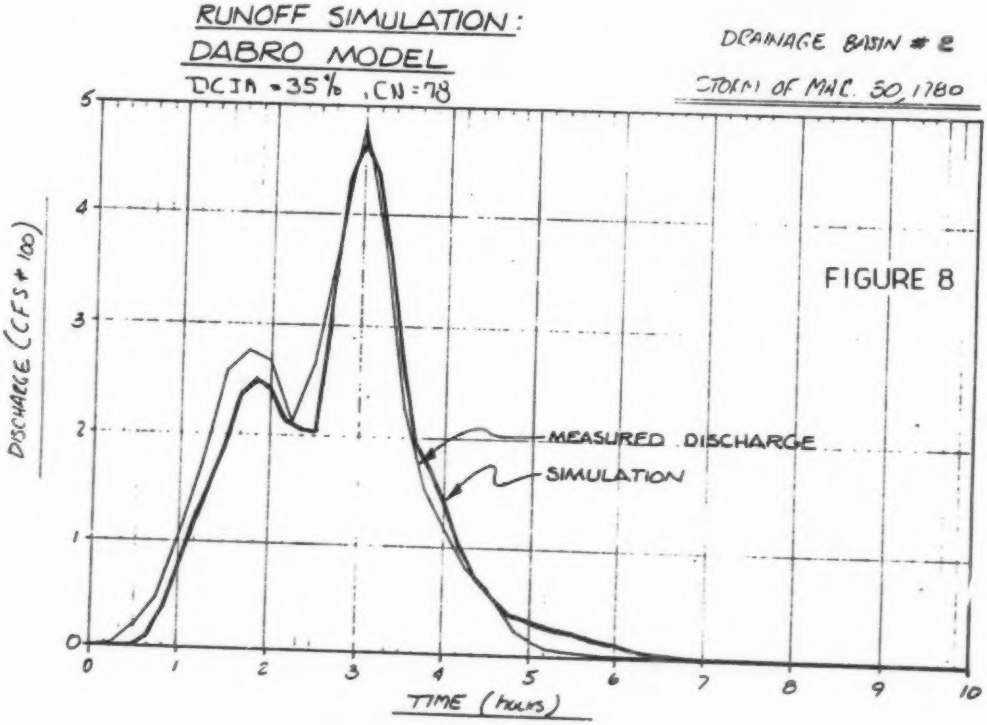


FIGURE 8

DRAINAGE BASIN # 2 STORM OF NOVEMBER 24, 1980
DABRO MODEL

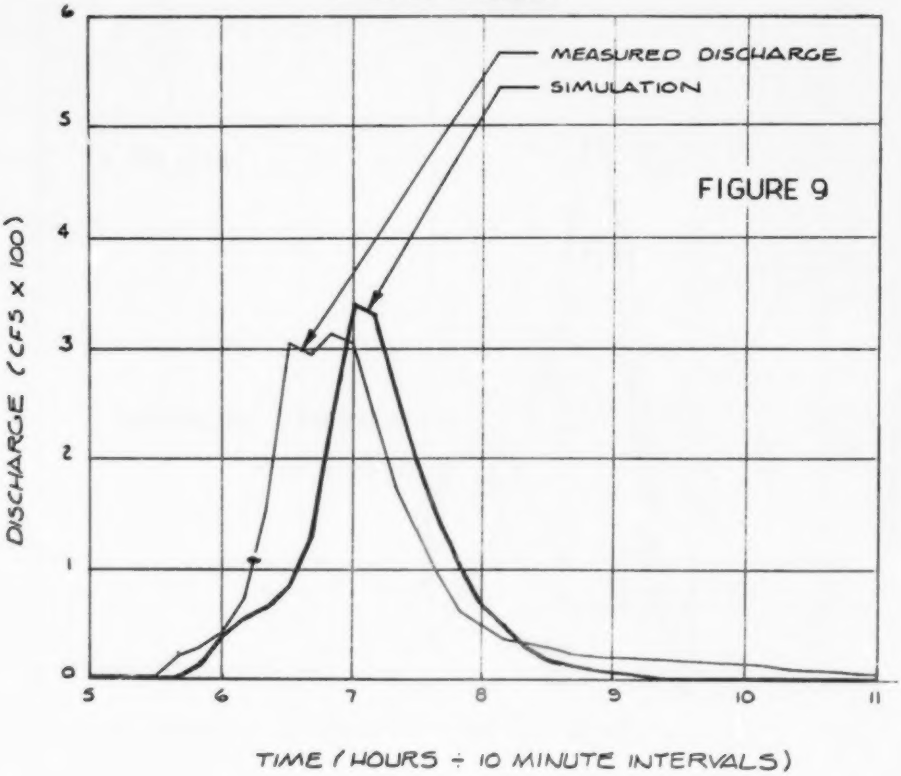


FIGURE 9

DRAINAGE BASIN # 2 STORM OF MARCH 30, 1981
DABRO MODEL

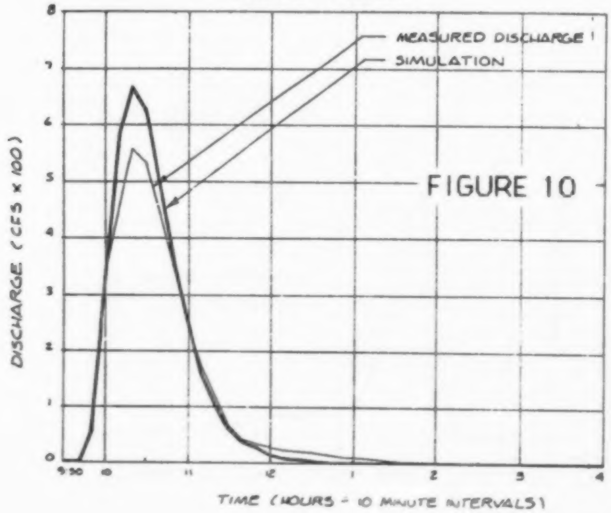


FIGURE 10

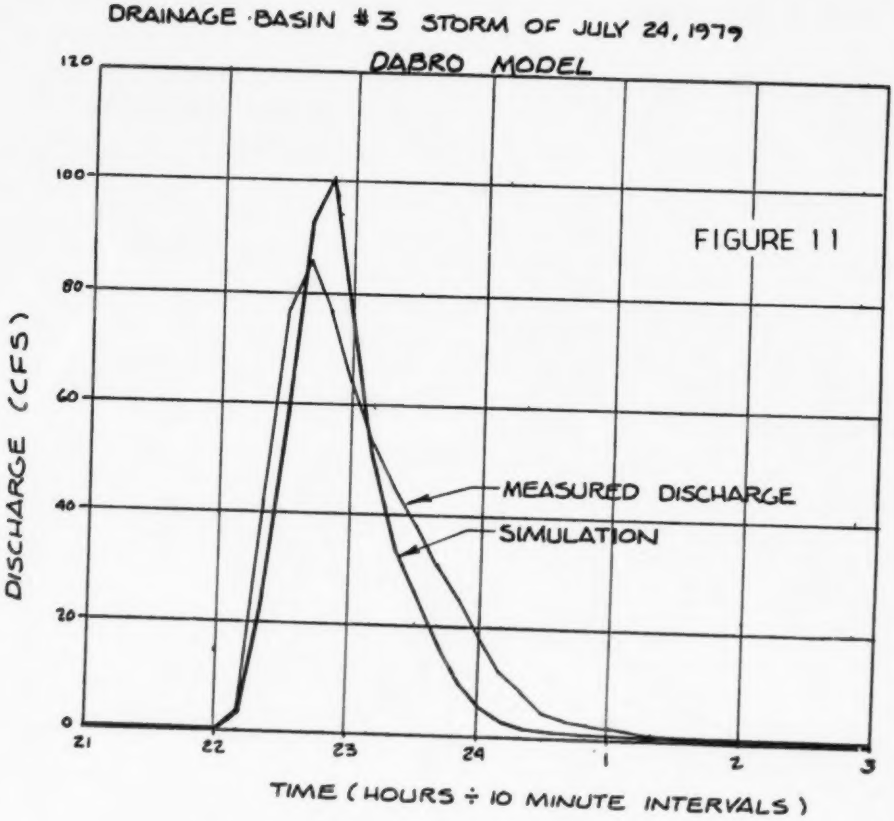


FIGURE 11

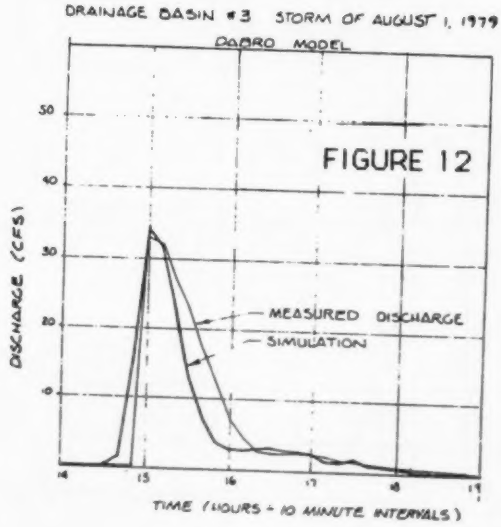


FIGURE 12

DRAINAGE BASIN #3 STORM OF SEPT 6, 1979
DABRO MODEL

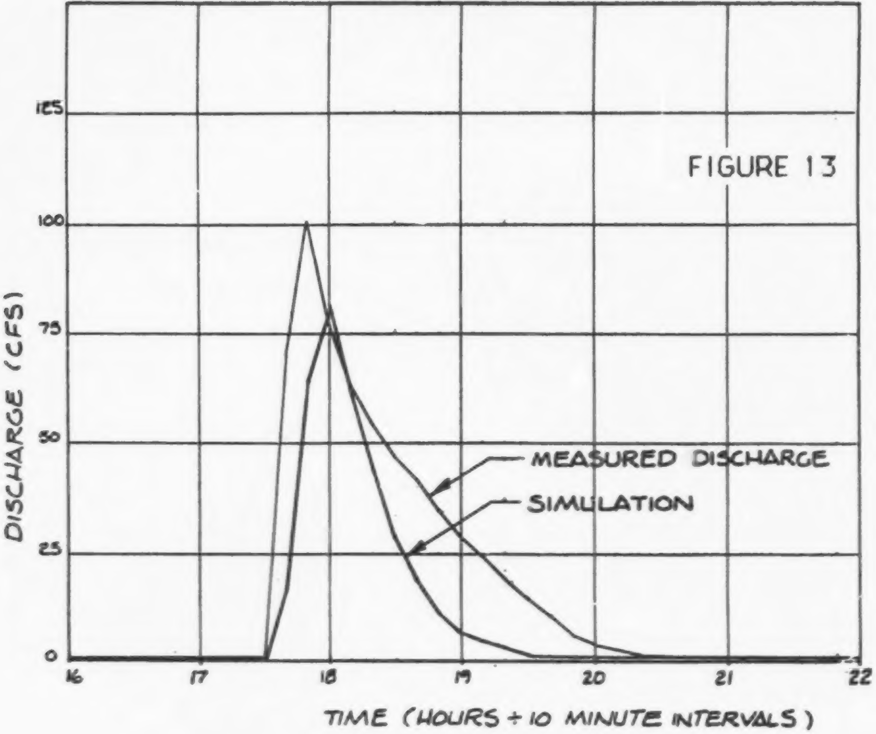


FIGURE 13

DRAINAGE BASIN # 3 STORM OF SEPT 21, 1979
DABRO MODEL

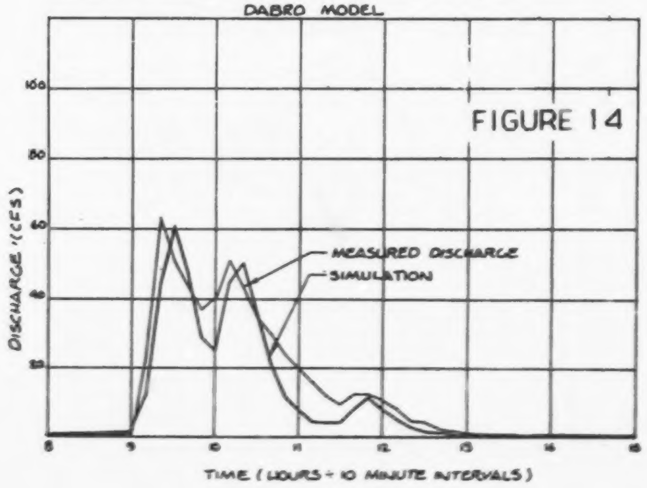


FIGURE 14

DRAINAGE BASIN #3 STORM OF MARCH 10, 1980

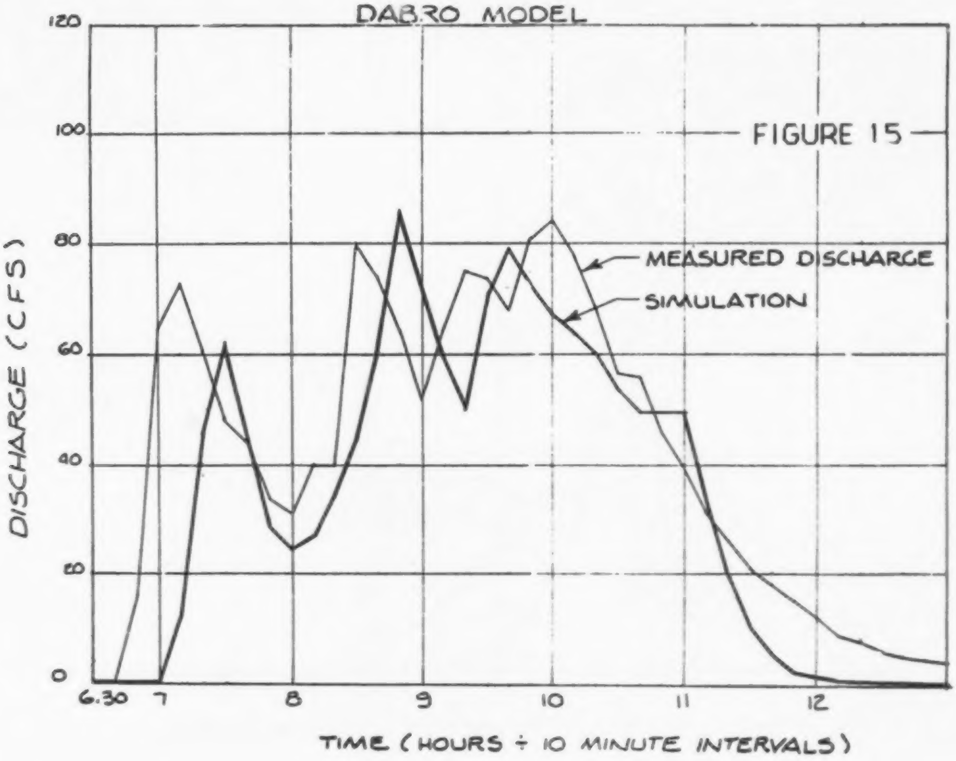


FIGURE 15

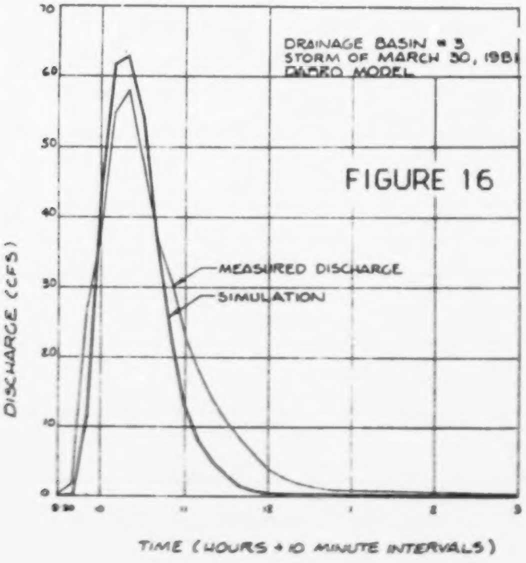


FIGURE 16

unit hydrograph duration D, ten-minute rainfall increments of the various events simulated were inputs to the program. No attempt was made to calibrate the model to improve the simulation results.

As can be observed from these Figures, the results of the simulations were quite good, except for the rainfall of March 31, 1981 on Basin No. 1, shown on Figure 5, in which case it appeared that the 1200 feet long, 8 feet diameter conduit under the FSU campus was inadequate to convey the peak runoff rate resulting from that particular rainfall.

Rural Basin: Simulation of various runoff events from the 130 square mile Manatee River basin (Basin No. 4) using the DABRO Model was done as part of a real-time reservoir regulation program implemented by Camp Dresser and McKee for the Lake Manatee Dam(5).

For the prior design of a new emergency spillway for the dam and for the purposes of a possible future real-time regulation program, the basin had been previously subdivided into five subbasins and runoff curve numbers based on land use in each of the subbasins for the AMC II condition computed. Also, as part of this prior work, two or more rainfall gages with telemetering equipment had been installed in each of the subbasins.

As part of the simulation work, the lag time of each subbasin was computed by Snyder's Method using a C_t=2.2 and the time to peak Tp (=D/2 + tp) for the two-hour unit hydrographs for each subbasin computed. The peak flow rate of each of the two-hour unit hydrographs were then computed by the equation Qp=CA/Tp. A "C" value ranging from 300 to 425 for each subbasin was selected based on the characteristics of the individual subbasin.

The results of simulation of the June 17-18, 1982 rainfall, obtained by summing and/or routing the various subbasin hydrographs, are shown on Figure No. 17. This was a fairly major rainfall event in that an average of approximately 6.6 inches of rainfall fell over the basin in a 38-hour period, most of which fell in a 24-hour period. This resulted in 2.96 inches of runoff at the dam. The prior dry period to this rainfall was

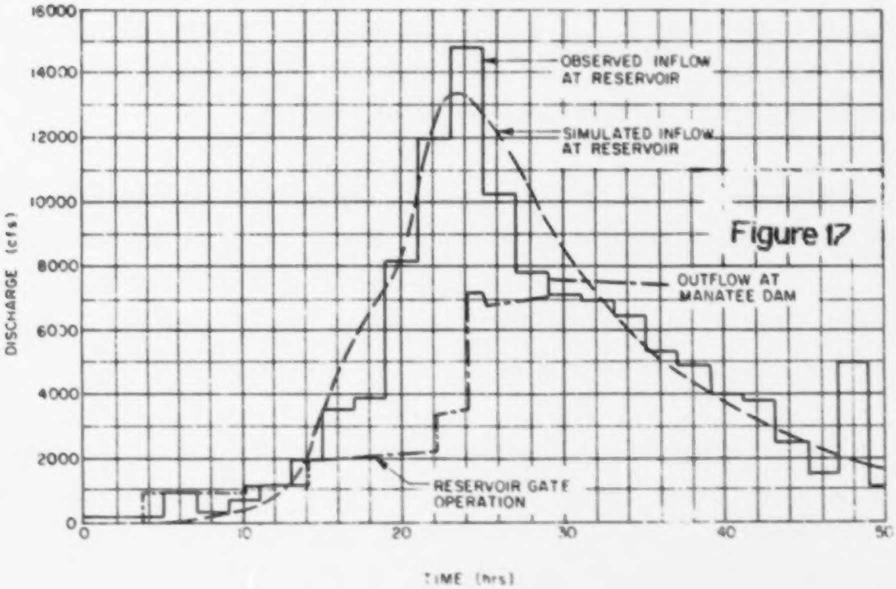


Figure 17

reflected by the average computed basinwide CN of 67. In the computation of the simulated inflow, shown on Figure 17, each of the previously computed subbasin CN's for AMC II were adjusted down three units to account for the difference between the originally computed average basinwide CN of 70 for the AMC II condition and the actual average basinwide CN of 67 for this rainfall event.

As can be observed from Figure 17, reasonably good simulation of the June 17-18, 1982 rainfall was achieved.

Verification of the various parameters input to the DABRO Model and the model itself were limited by the lack of a second major rainfall event. However, reasonable verification was achieved using the Storm of August 28, 1981 during which an average of 2.3 inches of rainfall fell over the basin resulting in 1.1 inches of runoff. This large runoff volume (CN=85) reflected the relatively large amount (3 inches) of rainfall which fell in the three days prior to the August 28, 1981 event. Again, the originally computed subbasin CN's for AMC II were adjusted (this time upward) to account for the difference between the computed average basinwide CN of 70 for the AMC II condition, and the actual average basinwide CN of 85. Note that this value of CN=85 is very close to the CN value for AMC III as given in Section 4, Hydrology of the SCS's National Engineering Handbook.

However, because of the obvious extreme importance of the antecedent moisture condition in this sandy basin, the following equation, based on the standard API, was derived to enable adjustment of the CN values in the Manatee River basin.

$$S = 0.48 + 6.72/API$$

where

S = average basin storage coefficient

In the application of the adjustment factor, the average basin storage coefficient, based on API, is first computed by Equation 6 and then the basinwide runoff curve number CN, based on the API, computed by the standard SCS equation. A basinwide correction factor, which is then applied to the runoff curve number of each subbasin, is then computed by the following equation:

$$CN = CN_{API}-70$$

Simulation of the August 28, 1981 event using these equations is shown on Figure 18.

Both of the above equations have been integrated into the DABRO Model and the Model revamped (DABRO 1) such that inputs to the model are now only rainfall increments and API.

ADVANTAGES OF THE DABRO MODEL

1. The DABRO Model is a simple, easy to use interactive model which instructs the user as to parameter input.

2. The model has been verified by application to both urban and rural basins. However, like all models, careful determination of subbasin parameters is necessary.

3. The model takes both the total impervious area and directly connected impervious area into consideration.

4. The unit hydrograph parameters (time to peak and peak flow rate of the various subbasins) can be changed by the user to fit the local situation.

5. The program, written in Microsoft Basic and not copy protected, is very easy to modify since almost any engineer can understand what the various lines in the program do. For instance, A=area, V=velocity, Q=flow rate, etc.

6. The program will operate on any microcomputer with 64K's of RAM. However, with a 64K computer, the program is limited to six subbasins.

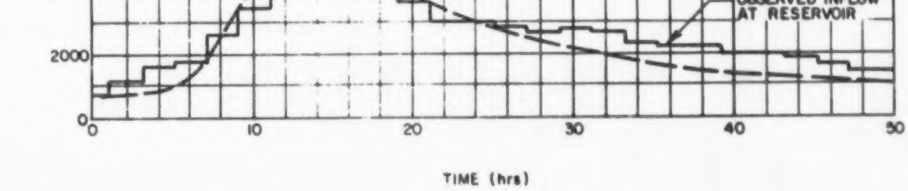


Figure 18

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1. "SCS National Engineering Handbook, Section 4, Hydrology", Soil Conservation Service, U. S. Department of Agriculture, Washington D.C., 1972.

2. Golding, Bernard L., Discussion of Paper Titled "Runoff Curve Numbers with Varying Site Moisture", Journal I and D Div., Proceedings ASCE, Vol. 105, IR4, December 1979.

3. Constant, James A., "A Mathematical Determination of the Ordinates of a Unit Hydrograph", Proceedings of the Seminar on Urban Hydrology, Hydrologic Engineering Center, Davis, California, September 1970.

4. Espey, W. H., Jr., Altman, D. G. and Graves, C. G., Jr., "Nomographs for Ten-Minute Unit Hydrographs for Small Urban Watersheds", ASCE Urban Water Resources Research Program, Technical Memorandum No. 32, ASCE New York, N.Y., December 1977.

5. Powell, Robert A., "Lake Manatee Reservoir Regulation Manual", Camp, Dresser and McKee, Inc., November 1983, 120 pp.

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APPLICATION OF A LOTUS SPREADSHEET FOR A SWMM PREPROCESSOR

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ABSTRACT

This report describes a preprocessor for the EPA Stormwater Management Model (SWMM) which is based on a Lotus 1-2-3 spreadsheet. Currently used methods to input data into SWMM are described and are compared to the Lotus-based preprocessor. The advantages and disadvantages of Lotus as a preprocessor host are discussed. The internal structure of the preprocessor commands as written in Lotus' macro language is described including the "Help" command which allows access to a number of reference files. Future improvements to the preprocessor are explained and conclusions as to its development are made.

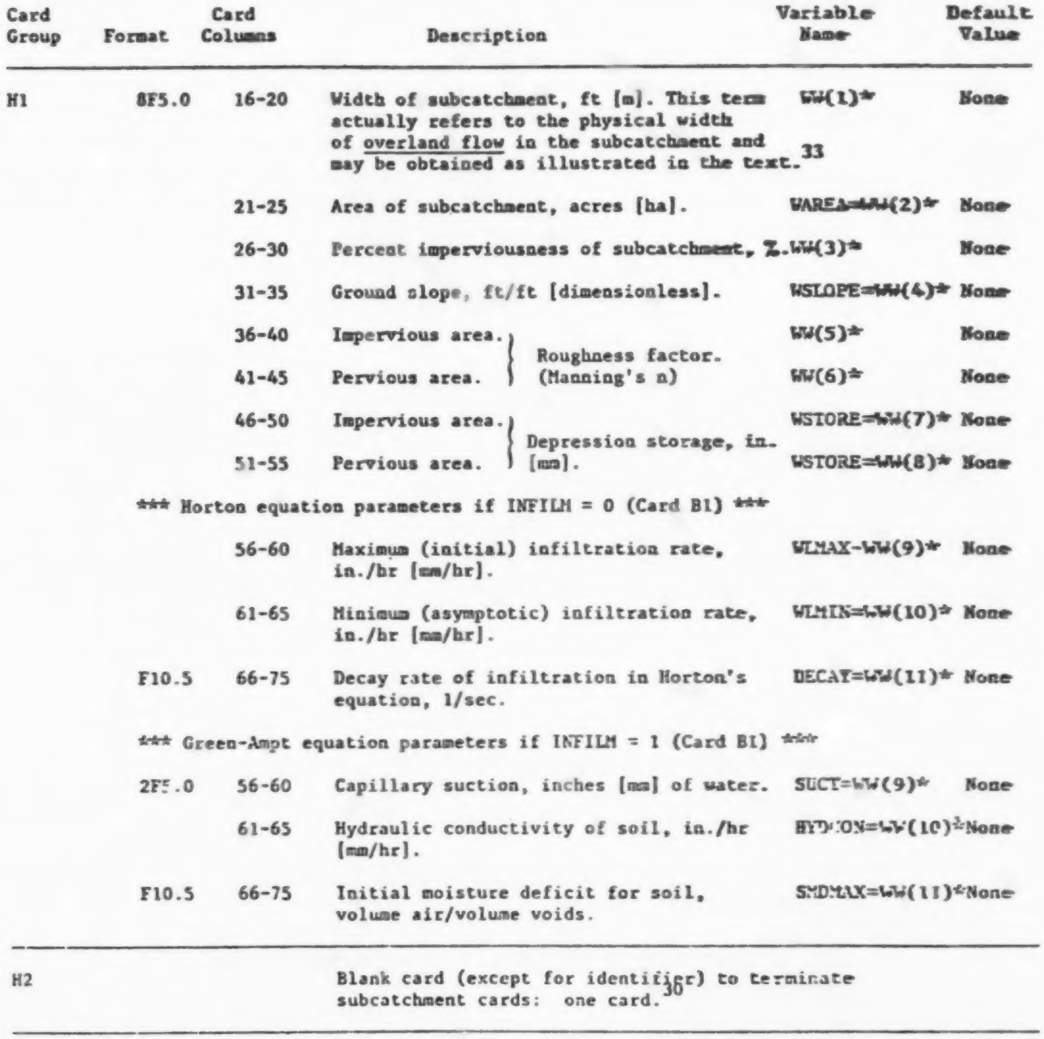
INTRODUCTION

Large mainframe computers have long been used to run hydrologic models. One of the most popular models, the EPA Stormwater Management Model (SWMM), has also been recently converted into a personal computer version available from several sources. This conversion has prompted the use of personal computers in many other areas of data preparation and handling in this field.

In order to fully exploit the potential of the personal computer, software packages may be used which create a structured format to store and handle data. This paper will describe one of these software packages, Lotus 1-2-3, and how it is being used to process input data for SWMM. The following sections in this paper will describe current methods of parameter entry into SWMM and will compare them with a Lotus 1-2-3 based preprocessor. The structure of each section of the preprocessor will also be described.

PARAMETER ENTRY IN ORIGINAL SWMM

The original method of parameter entry into SWMM was very tedious. The model user had to enter all parameters into the input file in the correct format. Each input card had to be entered in the correct order and each parameter had to be entered in the correct space allotments within each card. The user was totally dependent on the written documentation to determine the correct format as is shown in the excerpt from the SWMM User's Manual (1984) in Figure 1. It was easy to make errors in entering values and difficult to determine where format errors had been made. Even though this method of parameter entry is still being used, it is quickly becoming obsolete.



Card Group	Format	Card Columns	Description	Variable Name	Default Value
H1	8F5.0	16-20	Width of subcatchment, ft [m]. This term actually refers to the physical width of overland flow in the subcatchment and may be obtained as illustrated in the text.[33]	WW(1)*	None
		21-25	Area of subcatchment, acres [ha].	WAREA=WW(2)*	None
		26-30	Percent imperviousness of subcatchment, %.	WW(3)*	None
		31-35	Ground slope, ft/ft [dimensionless].	WSLOPE=WW(4)*	None
		36-40	Impervious area. Roughness factor. (Manning's n)	WW(5)*	None
		41-45	Pervious area.	WW(6)*	None
		46-50	Impervious area. Depression storage, in. [mm].	WSTORE=WW(7)*	None
		51-55	Pervious area.	WSTORE=WW(8)*	None
	*** Horton equation parameters if INFILM = 0 (Card B1) ***				
		56-60	Maximum (initial) infiltration rate, in./hr [mm/hr].	WLMAX-WW(9)*	None
		61-65	Minimum (asymptotic) infiltration rate, in./hr [mm/hr].	WLMIN=WW(10)*	None
	F10.5	66-75	Decay rate of infiltration in Horton's equation, 1/sec.	DECAY=WW(11)*	None
	*** Green-Ampt equation parameters if INFILM = 1 (Card B1) ***				
	2F5.0	56-60	Capillary suction, inches [mm] of water.	SUCT=WW(9)*	None
		61-65	Hydraulic conductivity of soil, in./hr [mm/hr].	HYDCON=WW(10)*	None
	F10.5	66-75	Initial moisture deficit for soil, volume air/volume voids.	SMDMAX=WW(11)*	None
H2			Blank card (except for identifier) to terminate subcatchment cards: one card.[30]		

Figure 1. Example of parameter format description in SWMM User's Manual (1984).

IMPROVED PARAMETER ENTRY INTO SWMM

Improvements on the manual parameter entry method have been made. In the personal computer version of SWMM currently being marketed by James and Robinson (1985), the user is prompted to enter parameter cards in their correct order (see Figure 2.). Prompts for the correct number of each card type are also included according to previously entered parameter values. The user is still responsible, however, for positioning the parameter values within each card. The user's manual must be consulted for each card entry as to the order in which the parameters are entered on each card and the default values for the parameter. This program is helpful in that the user does not need to enter correct spacing between each parameter on a card. The values need only to be separated by commas. If a card is entered erroneously, however, this program relies on outside editing procedures. Once the card entry process begins, it must be completed. Then the file may be edited with a standard disk operating system. This program is clearly an improvement over the original SWMM input procedure.

MENU-DRIVEN PARAMETER ENTRY

Another approach to this problem is used by the South Florida Water Management District (SFWMD). They have been distributing a menu driven program which aids in performing calculations needed in their permitting procedures. The program is menu driven in that the user is offered choices throughout the program. Initially, the user may choose from the Soil Conservation Method, the Santa Barbara Method, or his own method (Mass Route) to perform runoff calculations (see Figure 3). This program is easy to use and contains convenient editing procedures. The program does lack, however, sufficient documentation for use without consulting the reference manual.

```
 E3.  ENTER SWMM BLOCK TO BE CALLED                   ...
  RUNOFF

 SWMM RUNOFF BLOCK REQUESTED

 ENTRY MADE TO SWMM RUNOFF BLOCK INPUT

 R1.  ENTER FIRST TITLE CARD (DATA GROUP A1)          ...
A1SFWMD REPORT EXAMPLE

 R2.  ENTER SECOND TITLE CARD (DATA GROUP A1)         ...
A1RUNOFF MODULE

 R3.  ENTER FIRST CONTROL CARD (DATA GROUP B1)        ...
B10,0,0,1,0,0,0,12,0,1,7,85,0,0,0

 R4.  ENTER SECOND CONTROL CARD (DATA GROUP B2)       ...
B260,60,30,0,0,0
```

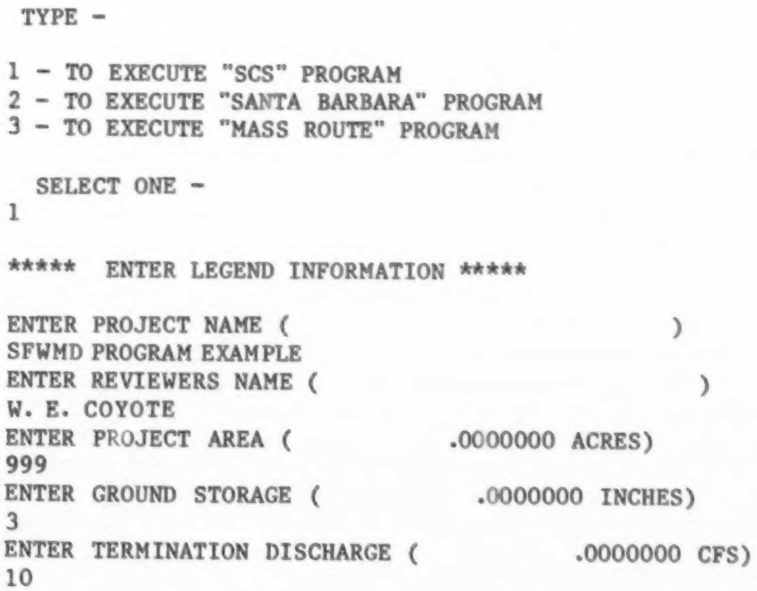
Figure 2. Example of parameter entry mode in PCSWMM (James and Robinson, 1985).

DESIRED STRUCTURE IN A SWMM PREPROCESSOR

In order to create a working SWMM input processing program, several ideas must be addressed. Firstly, all of the problems previously mentioned with current parameter entry methods must be solved. The program should include a format with which the user may input all parameters directly into a space allotted specifically for that parameter. This would necessitate a program with clear documentation. The user should also have free editing capabilities within the program. The user should be able to move back and forth between cards while entering parameter values, and change any previously entered values while continuing the current parameter entry mode. Also the choice of the values themselves is important. The user must feel comfortable with the choice of each value. Therefore, a complete program should include documentation which aids the user in choosing parameter values. If calculations are needed to choose any value, a format for these to be performed should be included. Lastly, this program should be able to operate such that the beginning user is comfortably led step by step while the experienced user is not slowed by the entry process.

OTHER PREPROCESSORS

A good example of a model preprocessing program was one for the CREAMS Model by Dennison and James (1985) using dBASE III. They present several ideas which are important in the development of a working preprocessor for a model: 1) It should be flexible and easy for a beginning user while not hindering expert users. 2) It should refer the user to the manual when additional



```
 TYPE -

1 - TO EXECUTE "SCS" PROGRAM
2 - TO EXECUTE "SANTA BARBARA" PROGRAM
3 - TO EXECUTE "MASS ROUTE" PROGRAM

  SELECT ONE -
1

*****  ENTER LEGEND INFORMATION *****

ENTER PROJECT NAME (                              )
SFWMD PROGRAM EXAMPLE
ENTER REVIEWERS NAME (                              )
W. E. COYOTE
ENTER PROJECT AREA (           .0000000 ACRES)
999
ENTER GROUND STORAGE (           .0000000 INCHES)
3
ENTER TERMINATION DISCHARGE (           .0000000 CFS)
10
```

Figure 3. Sample from South Florida Water Management District runoff and flow routing program with menu choices (SFWMD, 1983).

information is needed. 3) It should provide a convenient edit mode. All of these ideas are necessary features for a complete preprocessor.

A choice of calculation procedures for the user is an important aspect of the South Florida Water Management District's program (1985). Their program generates and/or routes hydrographs. The beginning of this program asks the user to choose from the Soil Conservation Service method, the Santa Barbara Unit Hydrograph method, or an alternative method for generating runoff hydrographs. This feature allows the user to generate hydrographs by whatever method he feels confident and comfortable. An interesting addition to a program such as this might be provisions for a "knowledge base" from which the chosen calculation procedure may extract information. In such a program, however, the user should be able to check all intermediate calculations and assumptions without having to review program lines whenever possible.

LOTUS 1-2-3 AS A PREPROCESSOR HOST

The idea of using a personal computer based system as a preprocessor for the EPA Stormwater Management Model is examined in this paper. Whether this system is used with a personal computer version of SWMM or interfaced with the mainframe version, the advantages of the personal desktop computer for parameter entry into the model are many. Among the software packages which are available for the personal computer, however, no clear favorite has been generally accepted to host a SWMM preprocessor. The many software options available all have their own advantages and disadvantages in terms of data entry, calculation, and documentation capabilities. This report will describe the mechanics and basic layout of a SWMM preprocessor which is housed on the Lotus 1-2-3 spreadsheet software package (Lotus Development Corp., 1984).

The Lotus 1-2-3 package was chosen for its flexibility in all of the areas needed for developing the preprocessor. As a spreadsheet Lotus is, in a sense, a large electronic notebook which has approximately 3250 pages, all of which may be randomly accessed at any time. This feature will allow the user of the preprocessor free movement during the parameter entry mode and will ease any editing of old parameter values which may be needed. This feature will also allow the user to access reference files which may be needed during the parameter entry mode.

Lotus 1-2-3 also has its own internal macro language which allows data manipulation to be automated. Since the main feature of the preprocessor is to format the model parameters and input data such that they are accepted by SWMM, Lotus' simple but comprehensive macro language is a convenient tool. This macro language allows for the creation of menus which facilitate the execution of the macro programs. These menus create an atmosphere in which the user may logically and systematically generate a SWMM input file while randomly accessing reference files, and finally, formatting the parameters and exporting the complete input file into a file to be read by SWMM.

Another point of flexibility with the Lotus package is its interaction with other types of files in the personal computer environment. Lotus can import other database and word processing files into its spreadsheet format as well as exporting text and numbers out of the worksheet. This feature promotes Lotus as a preprocessor to be interfaced with other software packages and allows the option of postprocessing SWMM output with the use of Lotus' graphical and statistical capabilities. Depending on the size of the SWMM output file, all or part of the file may be imported onto a Lotus worksheet to be summarized, analyzed, and/or graphed.

Unfortunately, the Lotus spreadsheet also has a few drawbacks for use as a SWMM preprocessor. One area which causes problems in the parameter formatting procedure is Lotus' policy for placing numbers in a column. For the visual purpose of keeping a space between columns of numbers, Lotus 1-2-3 does not allow an entire column width to be filled with the digits of a number. A column with a width of five, therefore, can only hold a four digit number. A five digit number entered into this column will cause a row of asterisks to be displayed across the cell width. This does not cause problems while performing calculations within Lotus since the cell will still contain the value entered; the value simply cannot be displayed. This does cause problems when exporting the SWMM file from the Lotus worksheet into an interface file as is done with the preprocessor. Lotus will export the display on the screen and, therefore, a row of asterisks instead of the parameter value. It is often necessary when entering SWMM parameters to use the full number of digits allotted to a parameter by the SWMM read procedure. This problem of space allocation may be solved within Lotus by handling the parameters as text would be handled. Lotus 1-2-3 allows any text entered into a cell to completely fill the cell and even extend beyond. Treating the parameter as text, however, could increase the processing time up to a few minutes for a large number of subcatchments and could also permit a number with too many digits to be entered into a formatted input card for SWMM.

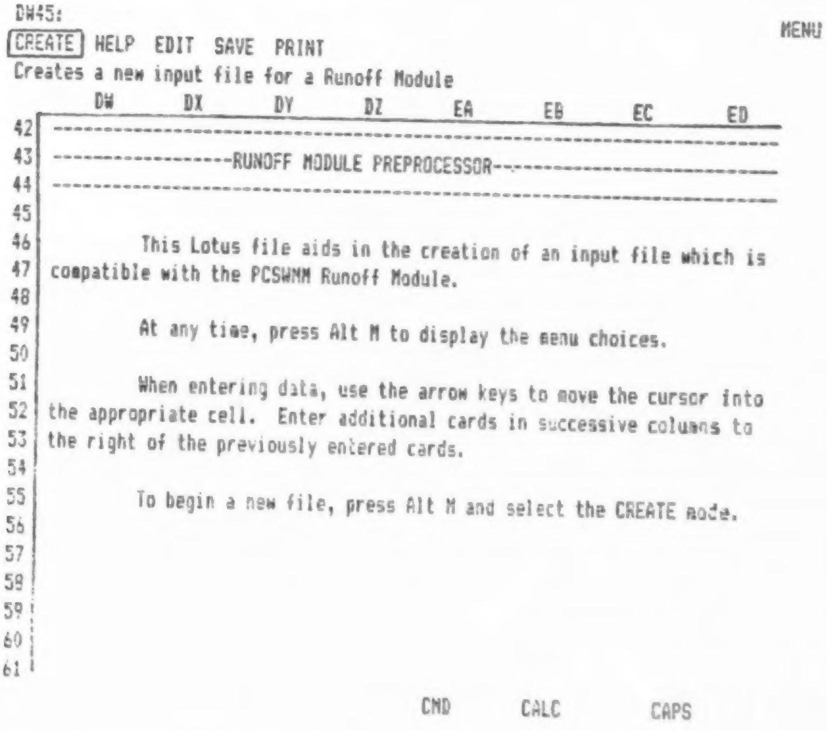
The spreadsheet format also seems, at this time, to impose certain size limitations on a preprocessor. These limitations will not come in terms of the number of cards (e.g. subcatchment) which may be entered into a module. The number of rain gauges and number of data points on a rainfall hyetograph may also be very extensive. The limitation is encountered because of the manner in which the parameters must be formatted with specific space allotments on a card. These allotments are provided by stringing card formats end to end along the top of the spreadsheet and adjusting column widths to receive each parameter. Cards are separated with different range names. Since the formatted cards span the entire width of the spreadsheet, this procedure limits the spreadsheet to only process one module per file. This in turn necessitates that each module be run singularly and that input, output, and interface files be called and manipulated manually. Manual file control is not difficult on small SWMM runs, but it can be cumbersome if larger runs employing several modules are performed.

MECHANICS OF A LOTUS PREPROCESSOR

The example file that has been created is a preprocessing package for a simplified version of the SWMM Runoff Module. In general, this preprocessor creates a structured work area in which the user may, in a systematic and orderly manner, enter the parameters necessary to run the Runoff Module. The sections and capabilities of this preprocessor are summarized and linked with a menu which was produced using the Lotus macro language. The menu allows the user to easily move from the parameter entry mode to the help/reference mode and access a number of help files. The menu also includes selections to execute the Lotus macro programs which will format the parameters into a Runoff input file and export this formatted file to an ASCII file. Once the user has loaded the package onto a Lotus spreadsheet, the introduction screen as shown in Figure 4 will appear. By pressing the keys Alt M, a macro program will begin which will display the menu choices ("Create" "Help" "Edit" "Save" "Print"). Each of these choices will be explained individually in the following sections.

Create

The "Create" mode allows the user to begin a new job file. The first page of this selection will ask for input such as job initials and a job number which will later be used to name the formatted file created by the preprocessor. Items such as the number of subcatchments and number of gutters in the simulation are also requested in order to allocate space within the worksheet. The user will next move to the parameter entry section of the preprocessor. As can be seen in the sample of this section shown in Figure 5,



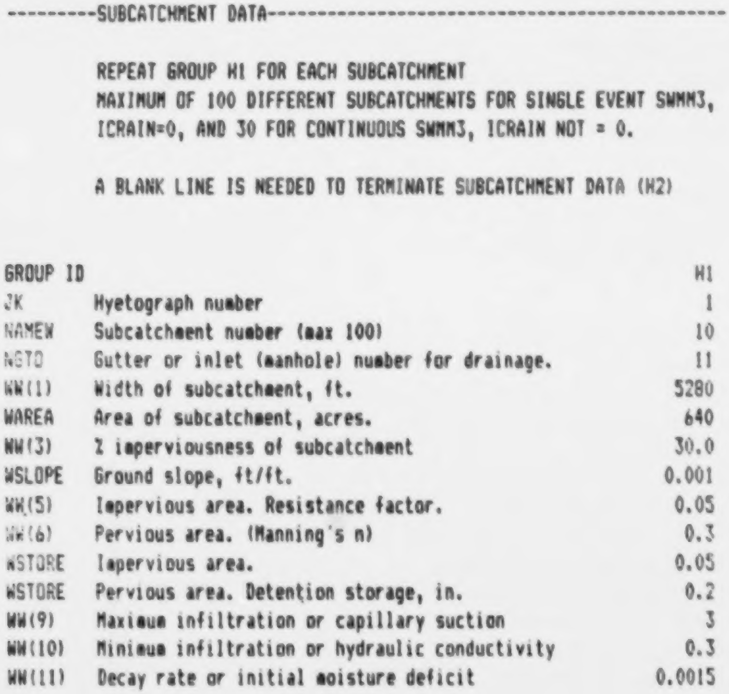
```
DW45:                                                                  MENU
[CREATE] HELP  EDIT  SAVE  PRINT
Creates a new input file for a Runoff Module
        DW        DX        DY        DZ        EA        EB        EC        ED
42 |-----------------------------------------------------------------------------
43 |-----------------RUNOFF MODULE PREPROCESSOR----------------------------------
44 |-----------------------------------------------------------------------------
45 |
46 |          This Lotus file aids in the creation of an input file which is
47 |compatible with the PCSWMM Runoff Module.
48 |
49 |          At any time, press Alt M to display the menu choices.
50 |
51 |          When entering data, use the arrow keys to move the cursor into
52 |the appropriate cell.  Enter additional cards in successive columns to
53 |the right of the previously entered cards.
54 |
55 |          To begin a new file, press Alt M and select the CREATE mode.
56 |
57 |
58 |
59 |
60 |
61 |
                                         CMD        CALC         CAPS
```

Figure 4. Introduction screen with menu choices displayed.

the parameter value is entered just to the right of a brief description of the parameter. The SWMM parameter name is also given at the far left of the screen. In cases where more than one of a given card type may be needed, such as with subcatchment cards, multiple card values are entered directly to the right of the initial card values. If the brief description of the parameter given in this section is not adequate to choose a value for a parameter or if calculations need to be performed to obtain a value, the user may access the help/reference section through the main menu as described in the next section.

Help

At any time during the parameter entry mode, the user may display the main menu by pressing Alt M, and then choose "Help" (see Figure 6). The user will then be prompted to enter a variable name whereupon a help file for that variable will be displayed, or to enter "index" whereupon the entire help file index will be displayed. From the help index the user will have access to help files for all parameters which will include further documentation, calculation aids, graphs, and any other material with which the user may make a logical and defensible parameter estimate for a specific job location. A



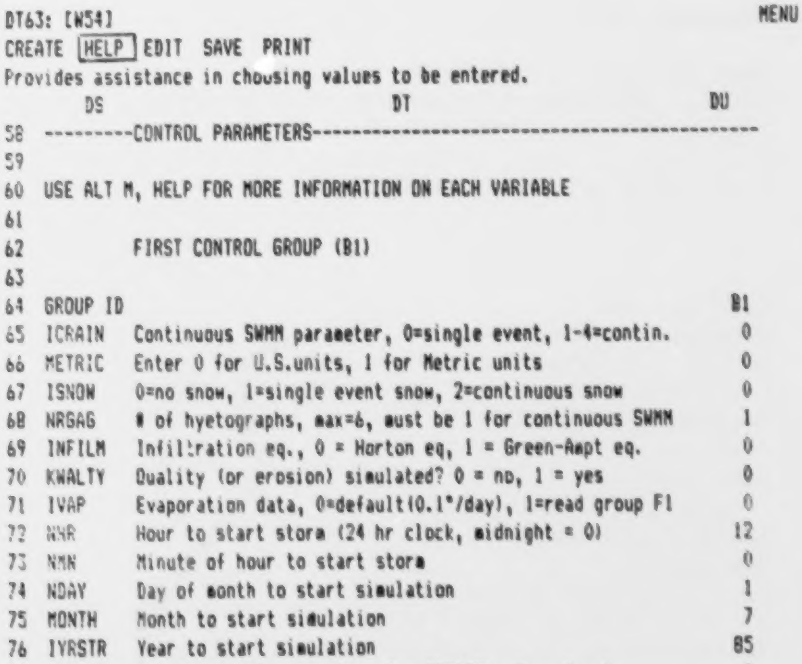
```
---------SUBCATCHMENT DATA--------------------------------------------

         REPEAT GROUP H1 FOR EACH SUBCATCHMENT
         MAXIMUM OF 100 DIFFERENT SUBCATCHMENTS FOR SINGLE EVENT SWMM3,
         ICRAIN=0, AND 30 FOR CONTINUOUS SWMM3, ICRAIN NOT = 0.

         A BLANK LINE IS NEEDED TO TERMINATE SUBCATCHMENT DATA (H2)


GROUP ID                                                           H1
JK       Hyetograph number                                          1
NAMEW    Subcatchment number (max 100)                             10
NGTO     Gutter or inlet (manhole) number for drainage.            11
WW(1)    Width of subcatchment, ft.                              5280
WAREA    Area of subcatchment, acres.                             640
WW(3)    % imperviousness of subcatchment                        30.0
WSLOPE   Ground slope, ft/ft.                                   0.001
WW(5)    Impervious area. Resistance factor.                     0.05
WW(6)    Pervious area. (Manning's n)                             0.3
WSTORE   Impervious area.                                        0.05
WSTORE   Pervious area. Detention storage, in.                    0.2
WW(9)    Maximum infiltration or capillary suction                  3
WW(10)   Minimum infiltration or hydraulic conductivity           0.3
WW(11)   Decay rate or initial moisture deficit                0.0015
```

Figure 5. SWMM Runoff module preprocessor, parameter entry mode.

documentation file may come directly from the manual as is the file shown in Figure 7. Once the user is finished viewing any specific help file, pressing return will display a menu offering the choices of returning to the parameter entry mode or returning to the help index. In returning to the parameter entry mode, the cursor is moved to the parameter for which the last help screen was viewed.

Edit

The edit mode is simply a short macro language program that takes advantage of Lotus' capability to move through the worksheet. This mode allows the user to update an old file which has already been created on the preprocessor, or to go back to a previous card on the present file and make a change. The user is able to move directly to any card in the module by choosing edit and moving the cursor to a card name as in Figure 8. A user more familiar with Lotus and the arrangement of SWMM cards will most likely be able to use the Lotus "goto" or "pageup/pagedown" keys to access a card more quickly than with this edit mode. The edit mode does, however, provide a list of the card types in their respective order for the less experienced user to choose from.

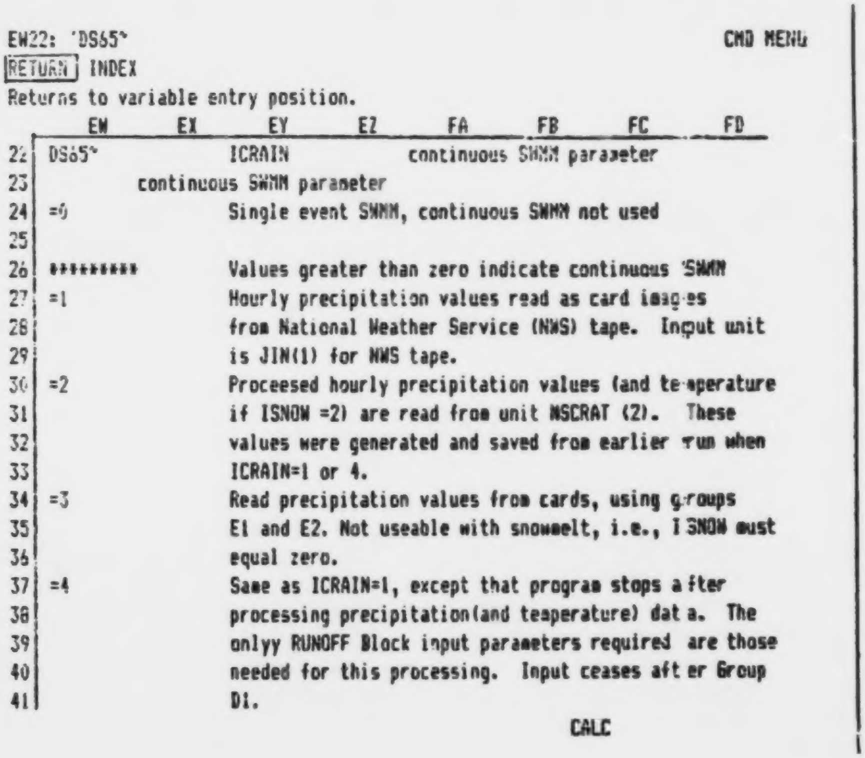


```
DT63: [W54]                                                        MENU
CREATE |HELP| EDIT  SAVE  PRINT
Provides assistance in chousing values to be entered.
      DS                          DT                          DU
58  ---------CONTROL PARAMETERS-------------------------------------------
59
60  USE ALT M, HELP FOR MORE INFORMATION ON EACH VARIABLE
61
62          FIRST CONTROL GROUP (B1)
63
64  GROUP ID                                                      B1
65  ICRAIN   Continuous SWMM parameter, 0=single event, 1-4=contin.    0
66  METRIC   Enter 0 for U.S.units, 1 for Metric units                 0
67  ISNOW    0=no snow, 1=single event snow, 2=continuous snow         0
68  NRGAG    # of hyetographs, max=6, must be 1 for continuous SWMM    1
69  INFILM   Infiltration eq., 0 = Horton eq, 1 = Green-Ampt eq.       0
70  KWALTY   Quality (or erosion) simulated? 0 = no, 1 = yes           0
71  IVAP     Evaporation data, 0=default(0.1"/day), 1=read group F1    0
72  NHR      Hour to start storm (24 hr clock, midnight = 0)          12
73  NMN      Minute of hour to start storm                             0
74  NDAY     Day of month to start simulation                          1
75  MONTH    Month to start simulation                                 7
76  IYRSTR   Year to start simulation                                 85
77  IRPRNT   Print control parameter, for ICRAIN = 1 or 4 only         0
                                        CMD      CALC
```

Figure 6. Referencing help file from parameter entry.

Save

The "Save" command, along with the "Print" command, do not initiate user convenience modes as did previously described commands, but are formatting procedures which must be performed to record the latest version of a parameter file. The "Save" command is the most time consuming macro program in its execution since it calls a series of macro subroutines which perform the formatting procedure. The basic idea of this procedure is to divide the aforementioned parameter cards, which were strung end to end across the top of the worksheet, and arrange them vertically in their correct order. As was described previously, the parameter cards were strung end to end so that each parameter could be entered in its own column whose width had been adjusted to accept the correct parameter format. In order to stack these parameter cards while preserving the correct spacing between parameter values, it was decided to combine all values on each card into a single worksheet cell. The task was performed by printing each range containing a card type separately into a temporary ASCII file, and then importing these files back into the worksheet in their proper order while stacking them vertically. This procedure is slightly tedious and takes about ninety seconds for a simple version of the Runoff Module. The time to perform this procedure will increase in proportion



```
EW22: 'DS65^                                                    CMD MENU
RETURN  INDEX
Returns to variable entry position.
        EW        EX        EY        EZ        FA        FB        FC        FD
22 | DS65^                 ICRAIN              continuous SWMM parameter
23 |           continuous SWMM parameter
24 | =0                    Single event SWMM, continuous SWMM not used
25 |
26 | *********             Values greater than zero indicate continuous 'SWMM
27 | =1                    Hourly precipitation values read as card images
28 |                       from National Weather Service (NWS) tape.  Input unit
29 |                       is JIN(1) for NWS tape.
30 | =2                    Proceesed hourly precipitation values (and temperature
31 |                       if ISNOW =2) are read from unit NSCRAT (2).  These
32 |                       values were generated and saved from earlier run when
33 |                       ICRAIN=1 or 4.
34 | =3                    Read precipitation values from cards, using groups
35 |                       E1 and E2. Not useable with snowmelt, i.e., ISNOW must
36 |                       equal zero.
37 | =4                    Same as ICRAIN=1, except that program stops after
38 |                       processing precipitation(and temperature) data.  The
39 |                       onlyy RUNOFF Block input parameters required  are those
40 |                       needed for this processing.  Input ceases after Group
41 |                       D1.
                                                      CALC
```

Figure 7. Example help file from manual with return menu.

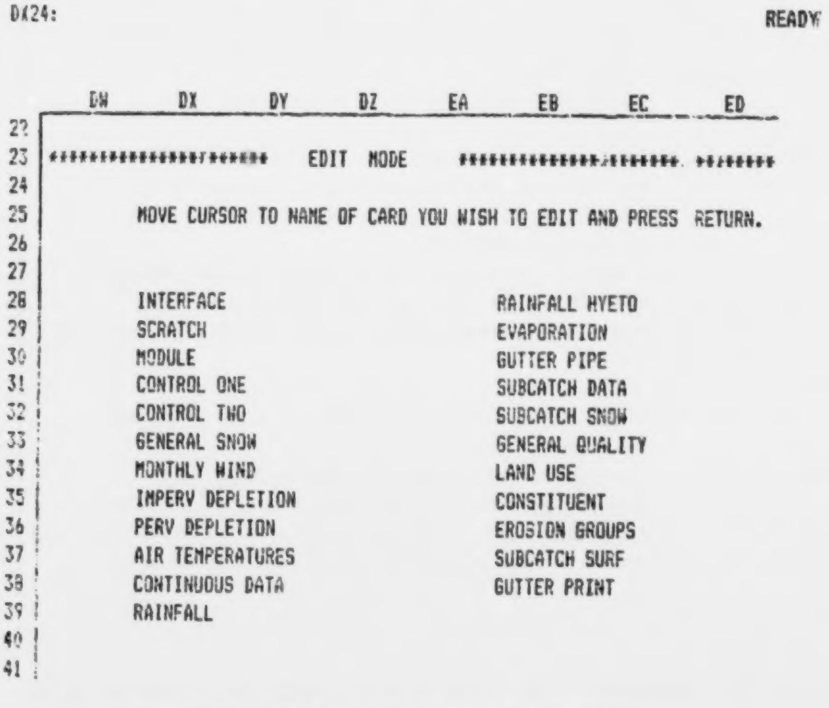
to the number of card types used, but not to the number of each card type used. For example, adding erosion cards and snow cards to a Runoff file would increase the run time by a few seconds each while adding an additional fifty subcatchment cards would add a minimal amount of run time.

Print

While the "Save" command is the workhorse of the preprocessor, the "Print" command takes all of the credit. Once the formatted listing of cards has been reviewed on the worksheet as seen in Figure 9, the user may invoke the "Print" command which will print the formatted cards from the worksheet into an ASCII file which may be accepted by a personal computer version of the Runoff Module as input. As mentioned earlier, the input file will be named using the job initials and job number entered at the beginning of the "Create" mode. Here again, however, a more experienced user may perform this print step manually on Lotus and provide another choice of file name.

STEPS TOWARD A COMPLETE PREPROCESSOR

The preprocessor which has been completed does not yet contain all cards which may be run with the Runoff Module. Though it can only format the parameters for a simple problem, it does contain all of the basic components

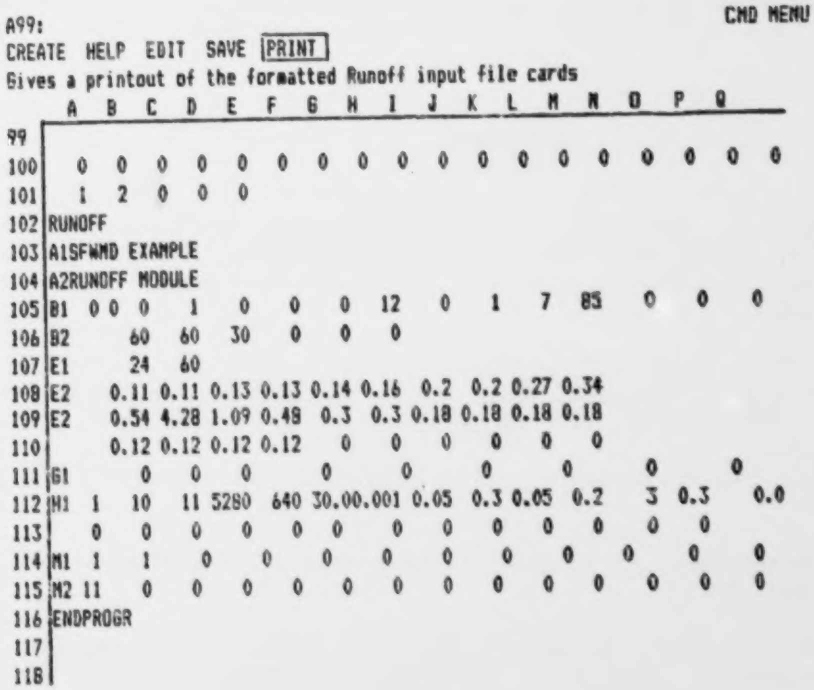


```
DX24:                                                        READY

        DW       DX       DY       DZ       EA       EB       EC       ED
   22 |
   23 |**********************    EDIT  MODE    ******************************
   24 |
   25 |      MOVE CURSOR TO NAME OF CARD YOU WISH TO EDIT AND PRESS  RETURN.
   26 |
   27 |
   28 |      INTERFACE                      RAINFALL HYETO
   29 |      SCRATCH                        EVAPORATION
   30 |      MODULE                         GUTTER PIPE
   31 |      CONTROL ONE                    SUBCATCH DATA
   32 |      CONTROL TWO                    SUBCATCH SNOW
   33 |      GENERAL SNOW                   GENERAL QUALITY
   34 |      MONTHLY WIND                   LAND USE
   35 |      IMPERV DEPLETION               CONSTITUENT
   36 |      PERV DEPLETION                 EROSION GROUPS
   37 |      AIR TEMPERATURES               SUBCATCH SURF
   38 |      CONTINUOUS DATA                GUTTER PRINT
   39 |      RAINFALL
   40 |
   41 |
```

Figure 8. List of card types in Edit mode.

needed in a preprocessor of this type. Once all of the card types have been introduced into the Runoff preprocessor, the only major step is to write a macro language program to delete cards that are not necessary for any particular run. Lotus provides all of the logistical functions which will be needed for this task and no problems are foreseen at this point in time.

The next step is to create a preprocessor for each SWMM module. Here again returns the limitation that only one module will be able to be introduced on a single worksheet preprocessor. Each preprocessor, however, will have the same basic skeleton as the Runoff Module preprocessor and should require less effort in its creation. Other improvements in this type of preprocessor may include providing a copy of the entire SWMM manual to be referenced through the help index. This would in theory free the user from leaving the personal computer to access information. The eventual interfacing of this preprocessor with an "expert system" would further this idea by providing the user with calculation alternatives to SWMM which may feed from a common "knowledge base" of information. This "knowledge base" may be thought of as a general source of data from which any model could extract necessary



```
A99:                                                              CMD MENU
CREATE  HELP  EDIT  SAVE  |PRINT|
Gives a printout of the formatted Runoff input file cards
      A    B    C    D    E    F    G    H    I    J    K    L    M    N    O    P    Q
 99 |
100 |   0    0    0    0    0    0    0    0    0    0    0    0    0    0    0    0    0    0
101 |   1    2    0    0    0
102 |RUNOFF
103 |A1SFWMD EXAMPLE
104 |A2RUNOFF MODULE
105 |B1  0 0  0    1     0     0     0    12     0     1     7    85     0     0     0
106 |B2       60   60    30     0     0     0
107 |E1       24   60
108 |E2     0.11 0.11 0.13 0.13 0.14 0.16  0.2  0.2 0.27 0.34
109 |E2     0.54 4.28 1.09 0.48  0.3  0.3 0.18 0.18 0.18 0.18
110 |       0.12 0.12 0.12 0.12    0    0    0    0    0    0
111 |G1        0    0    0       0       0       0       0       0       0
112 |H1   1    10   11 5280  640 30.00.001 0.05  0.3 0.05  0.2    3  0.3    0.0
113 |     0    0    0    0    0   0     0    0    0    0    0    0    0
114 |M1   1    1     0     0     0     0     0     0     0     0     0     0
115 |M2 11     0    0    0    0    0    0    0    0    0    0    0    0    0
116 |ENDPROGR
117 |
118 |
```

Figure 9. Formatted version of parameter cards.

information for its simulation. Random access to different methods of parameter calculation and years of data history through the help index would all lead toward rendering the user independent of conventional referencing methods.

CONCLUSIONS

1. The Lotus 1-2-3 software package is a suitable host for an input file/reference file preprocessing package that can be interfaced with the EPA Stormwater Management Model (SWMM).

2. The Runoff Module preprocessor which has been created is a simplified package, but it reflects well a basic skeleton which may be used to create preprocessing packages for the remaining modules of SWMM. No technical difficulties in creating these packages on Lotus spreadsheets are foreseen.

3. The preprocessor idea may be the first step toward rendering the personal computer user independent from conventional referencing techniques.

4. The "Create", "Help", and "Edit" modes of this preprocessor are sufficiently documented for easy use by a beginner with a minimal knowledge of Lotus 1-2-3 and SWMM, but do not hinder their use by an expert.

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IMPACT OF EXTENSIVE IRRIGATION PUMPAGE ON STREAMFLOW BY HSPF

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ABSTRACT

The hydrology of a 2,700 square mile area of the Big Blue River Basin in central Nebraska was simulated by the continuous process hydrologic simulation program HSPF to investigate the impacts of extensive groundwater pumpage for irrigation on streamflows. A long-term continuous simulation for a 23-year period (1953-1975) was carried out by modeling water movement and storage characteristics through the use of the PWATER section of PERLND module and HYDR section of the RCHRES module of HSPF. The simulation indicated that surface runoff and interflow from irrigated lands increase initially. Actual evapotranspiration also increases. As pumping continues and the aquifer water level drops, groundwater contribution to streamflow decreases. Finally, a stable condition is achieved where the water table ceases to reach the stream bed and measured streamflow is less than before pumping began. It is concluded that HSPF can be used with a certain degree of success to simulate stream-aquifer interaction and to assess possible effects of deep pumping on surface water hydrology in a Nebraska basin. The accuracy of the results, however, is limited by modeling complications because some algorithims of HSPF were not specifically designed to simulate withdrawal of groundwater by pumping. Coordination of some HSPF input parameters from groundwater modeling may produce better results.

INTRODUCTION

Development of irrigation using groundwater started growing rapidly in the 1950's and expanded very rapidly in the 1970's in Nebraska. By 1980, more than 73 percent of water used for irrigation in the state was pumped from wells. This development, however, has not been without consequences. The impact of extensive groundwater pumpage for irrigation was noticeable in the flow regimen of several streams, including the Big Blue River basin in central Nebraska. As a part of the Nebraska Natural Resources Commission's (NRC) "Problem Analysis and Area Planning" activity for the Big Blue River Basin, an attempt was made to investigate the impacts of groundwater pumpage on the streamflows of the Big Blue River and its tributaries using HSPF. This paper summarizes the methodologies used in simulating by HSPF the effect of the time history of groundwater pumpage on the hydrology of the basin, and the results obtained from the simulation. It also evaluates the applicability of HSPF in simulating deep groundwater pumpage and resulting groundwater-surface water interrelationships.

DESCRIPTION OF THE STUDY AREA

The Big Blue River Basin is located in south-central Nebraska and northeast Kansas, and is a part of Kansas River Basin. A map of the basin with principal tributaries is shown in Figure 1. The area of the Nebraska portion of the basin totals about 4,600 square miles. The topography ranges from loess plains with thick deposits of silt, sand, and gravel, to rolling hills and stream valleys.

The principal aquifer consists of unconsolidated sand and gravel layers of Pleistocene age. These layers belong to three formations composed of sand and gravel graded upward from coarser to finer material. Soils are of loess origin. About sixty percent of the soils in the basin are of irrigation suitability type A (i.e., with slight limitations only).

Land use is primarily agricultural. About 83 percent of agricultural lands are croplands. Since the early 1950's the number of irrigated acres using groundwater has increased every year. By 1980, approximately 1,061,000 acres in the basin were irrigated. The greatest concentration of irrigation wells is in four counties in the upper part of the basin. This intensive groundwater development, coupled with occassional drought conditions, has caused widespread water level declines. Water levels have declined as much as 30 feet in several areas.

Upper reaches of the streams are characterized by small meandering channels with intermittent flows. The streamflow is variable, being primarily derived from precipitation runoff. Nearly all of the tributaries are intermittent. Even the main stem of the Big Blue River does not become a perennial stream until it drains about 450 square miles of the upper part of the basin. Base flows in the streams are, however, relatively low. Return flows from increased well irrigation have lengthened the period of time when flows are present in many of the streams.

SELECTION OF SIMULATION MODEL

As a part of the problem analysis and area planning study of the basin water supply, a two-dimensional finite difference groundwater model covering most of the basin was developed. A large data base including irrigated lands identified by remote-sensing, geologic data, and groundwater conditions was generated for development of the groundwater model. However, since the groundwater model could not accurately represent the effects of the continuous changes on the surface water hydrology, it appeared that a long-term continuous model was needed to simulate the hydrologic-hydraulic behavior of the stream aquifer system prior to and during the period of extensive irrigation pumpage. A search was made for an integrated and comprehensive surface water hydrology model that would allow comparison of water movement, storage characteristics and hydrographs for conditions before and after development of irrigation. Based upon these requirements, the program package Hydrologic Simulation

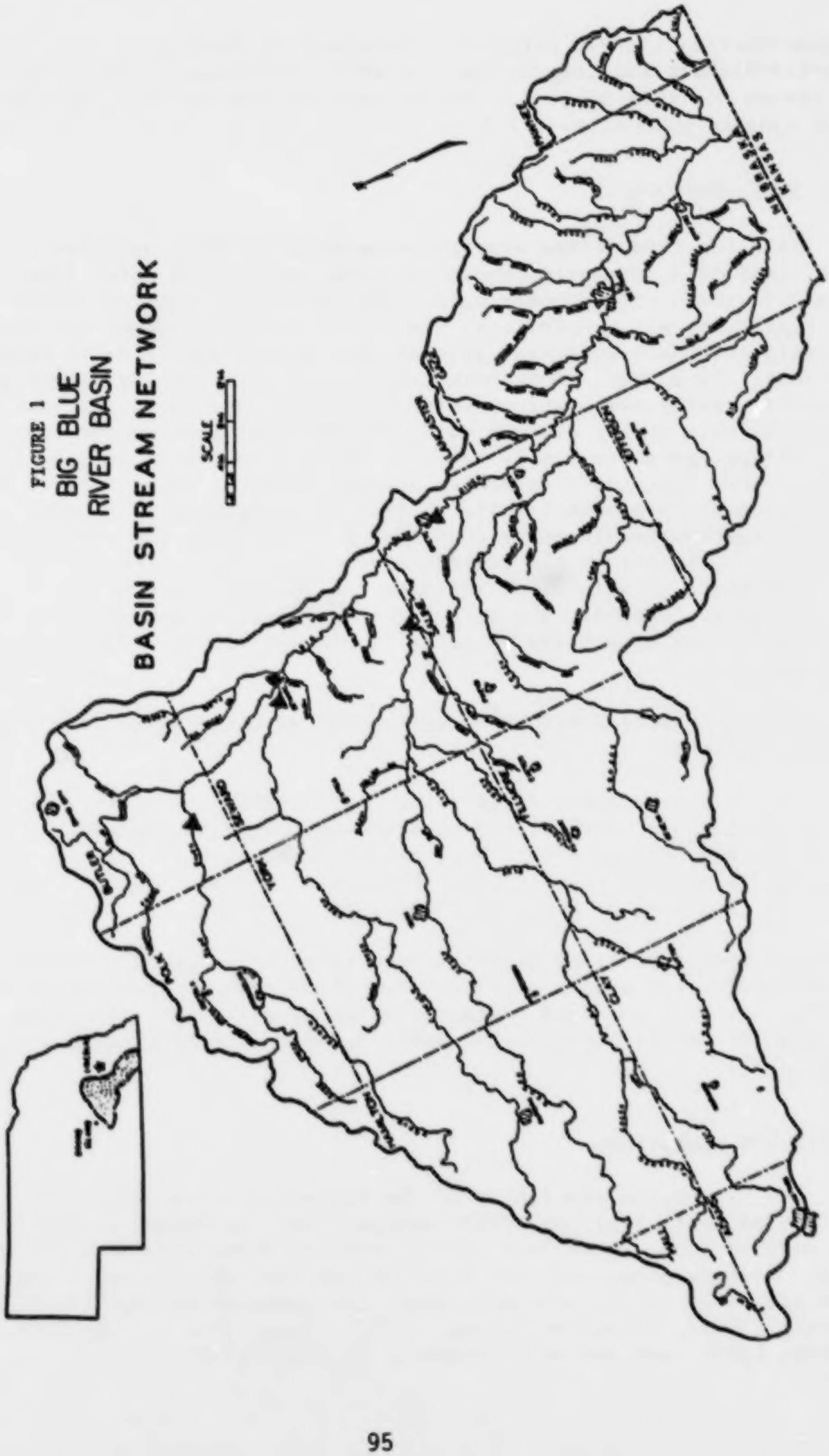

SCALE
NEBRASKA
KANSAS

FIGURE 1
BIG BLUE
RIVER BASIN
BASIN STREAM NETWORK

Program-Fortran (HSPF), originally developed by Hydrocomp, Inc. and revised by Anderson-Nichols and Company for the EPA's Environmental Research Laboratory, was chosen for this project. The project was started with Release 5.0, and later updated with Release 7.0.

DATA BASE DEVELOPMENT

Data for time series storage management of HSPF consisted of meteorological data, land data, channel-flood plain data, streamflow data, riverine structure data and groundwater pumpage data. Meteorological data consisted of precipitation, air temperature, dew point, solar radiation and wind movement. Precipitation data were available at five hourly and 21 daily recording stations. Daily maximum-minimum temperature data were available at three stations. Daily evaporation, solar radiation, dew point and wind movement data were available at one station each. Daily precipitation and temperature records were disaggregated to hourly values. Daily groundwater pumpage data were incorporated into the time series storage from the input data of the groundwater model already developed for the basin. In the groundwater model, the seasonal Crop Irrigation Requirements (CIR) for various crops and soil types in each node were first computed by the Jensen-Haise methodology. The CIR was then adjusted for irrigation efficiency to estimate the seasonal pumpage and distributed over the irrigation months. The acre inches per acre of pumpage from the GW model nodes were then disaggregated to daily values over the PERLND segments of the HSPF model.

Daily streamflow data for calibration were available at the following USGS gaging stations:

1) Big Blue River at Surprise (April 1964 - present)
2) Lincoln Creek near Seward (October 1953 - present)
3) Big Blue at Seward (October 1953 - present)
4) West Fork Big Blue near Dorchester (October 1958 - present)
5) Big Blue at Crete (October 1954 - Present)

Pervious hydrologic land segments were selected based on the unique combination of meteorological characteristics identified by a Thiessen polygon network of weather stations and land characteristics by soil type, topography and cover. A total of 38 pervious hydrologic land segments were delineated for simulation (Figure 2).

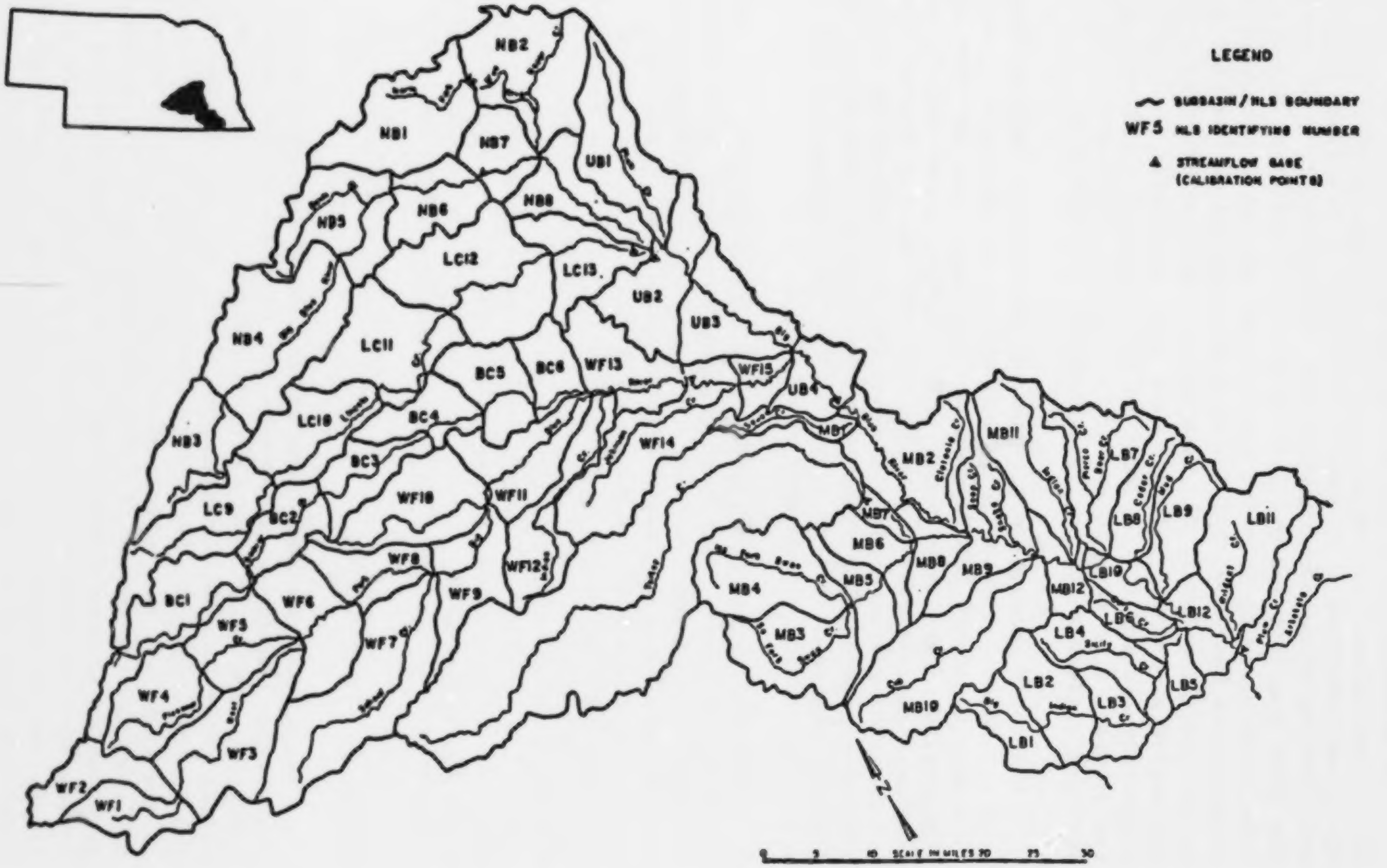
SIMULATION ALGORITHMS

The hydrologic simulation of the basin was conducted by the utilization of the PWATER section of the PERLND module. Water movement in the PERLND module was modeled along three flow paths: overland flow, interflow and groundwater flow. Storage processes that occur on the land surface and in the soil horizons were simulated in six storage blocks: Interception Storage (CEPS), Surface Storage (SURS), Interflow Storage (IFWS), Upper Zone Storage (UZS), Lower Zone Storage (LZS), and Active Groundwater Storage (AGWS).

Figure 2

BIG BLUE RIVER BASIN

HYDROLOGIC LAND SEGMENTS
INCORPORATED IN THE BIG BLUE
WATER SUPPLY MODEL


LEGEND
SUBBASIN/HLS BOUNDARY
WF5 HLS IDENTIFYING NUMBER
▲ STREAMFLOW GAGE (CALIBRATION POINTS)
NB1
NB2
NB3
NB4
NB5
NB6
NB7
NB8
UB1
UB2
UB3
UB4
LC9
LC10
LC11
LC12
LC13
BC1
BC2
BC3
BC4
BC5
BC6
WF1
WF2
WF3
WF4
WF5
WF6
WF7
WF8
WF9
WF10
WF11
WF12
WF13
WF14
WF15
MB1
MB2
MB3
MB4
MB5
MB6
MB7
MB8
MB9
MB10
MB11
MB12
LB1
LB2
LB3
LB4
LB5
LB6
LB7
LB8
LB9
LB10
LB11
LB12
N
0 5 10 SCALE IN MILES 20 25 30

The processes of infiltration and overland flow are interdependent and occur simultaneously. Water in surface detention will later infiltrate and appear as interflow or it can be contained in the upper zone storage. Water infiltrating through the surface and percolating from the upper zone storage to the lower zone storage may flow to active groundwater storage, or may be lost by deep percolation to inactive groundwater storage. Active groundwater eventually reappears as baseflow, but the deep percolation to inactive groundwater storage is not accounted for in the simulation. Lateral external inflows to interflow and active groundwater storage were also considered in simulation. Evapotranspiration is simulated in all phases of the storages associated with the process, i.e. from interception storage, upper and lower zone storages, active groundwater storage, and directly from baseflow.

Hydraulic simulation for routing runoff from land surface and discharge from groundwater though the stream system was performed by utilization of the HYDR section of the RCHRES module. Simulation reaches were identified based on homogeneity of cross-sectional shape, channel slope and channel-floodplain roughness coefficients. Detailed surveyed data on channel-floodplain cross sections and riverine structures on the main stem of Big Blue and West Fork Big Blue were available from the Natural Resources Commission Flood Plain Studies. Stage-discharge information computed by HEC-2 program in NRC Flood Plain studies were also utilized in generating FTABLES.

In applying the model on this particular project, the basic fluxes without and with the dynamic effect of groundwater pumpage on streamflow are illustrated in Figure 3. Streamflow comes from surface runoff and interflow, controlled by soil moisture storage, and from groundwater flow. For the conditions prior to extensive irrigation development, the solid line linkages of Figure 3 represent the hydrologic processes that existed in the basin. When irrigation pumpage was introduced, another flow path was added, which is shown as the dashed line of Figure 3. This flow path can be represented by various ways in HSPF, but the dashed line in Figure 3 seemed to be a good approximation of actual conditions. The layout of a typical pervious land segment incorporated into the Big Blue model to represent the interrelationship of surface and groundwater flow components under groundwater pumpage condition is illustrated in Figure 4.

METHOD OF INVESTIGATION

The primary strategy used in this study to evaluate the impact of deep groundwater pumpage on streamflow was to simulate the long-term hydrologic-hydraulic behavior of the basin for the conditions prevailing prior to the irrigation development conditions, and project that simulation through the period undergoing irrigation development. The hydrologic-hydraulic behavior of the basin was then simulated for the actual conditions prevailing in the basin within the period of irrigation development by incorporating the groundwater pumpage element into the model. Superimposing the series of hydrographs representing pre-irrigation conditions on the series representing irrigation development conditions would show the continual effect of streamflow resulting from the extensive withdrawal of groundwater.

CALIBRATION

Many of the algorithms contained in the model are mathematical approximations of complex natural phenomena. Before the model could be used to reliably simulate streamflow behavior under various conditions, it was necessary to calibrate the model by comparing simulation results with measured historical data. If significant differences were found, parameters were adjusted to calibrate the model to the specific natural and man-made features of the basin. The model was calibrated for the conditions prevailing during two periods. The first was prior to extensive groundwater pumpage, and the second was during the period of the greatest irrigation development.

Though the development of irrigation in the basin began to take place in 1948, total development was minor compared to that in the late 1960's and the 1970's. Also, available continuous streamflow records for almost all of the gages in the Big Blue basin do not start before water year 1954, preventing calibration of the model prior to 1954. It was, therefore, decided to use several periods between 1954 and 1960 for pre-irrigation development conditions, and periods between 1960 and 1975 for irrigation development conditions to calibrate the models of different stream networks, depending on the avilability of measured streamflow records for that stream. The stream segment for the Big Blue at Surprise, however, could not be calibrated for pre-irrigation development conditions as recorded streamflow data at this gage are available from only after October 1964. The months for which the model was calibrated are as follows:

1.	Big Blue at Surprise:	October 1966-September 1969
2.	Big Blue at Seward:	October 1953-September 1955 and October 1966-September 1968

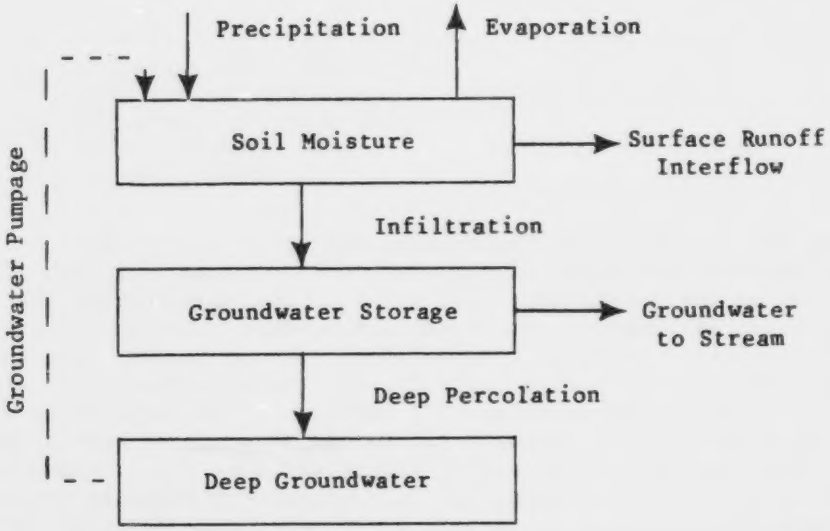
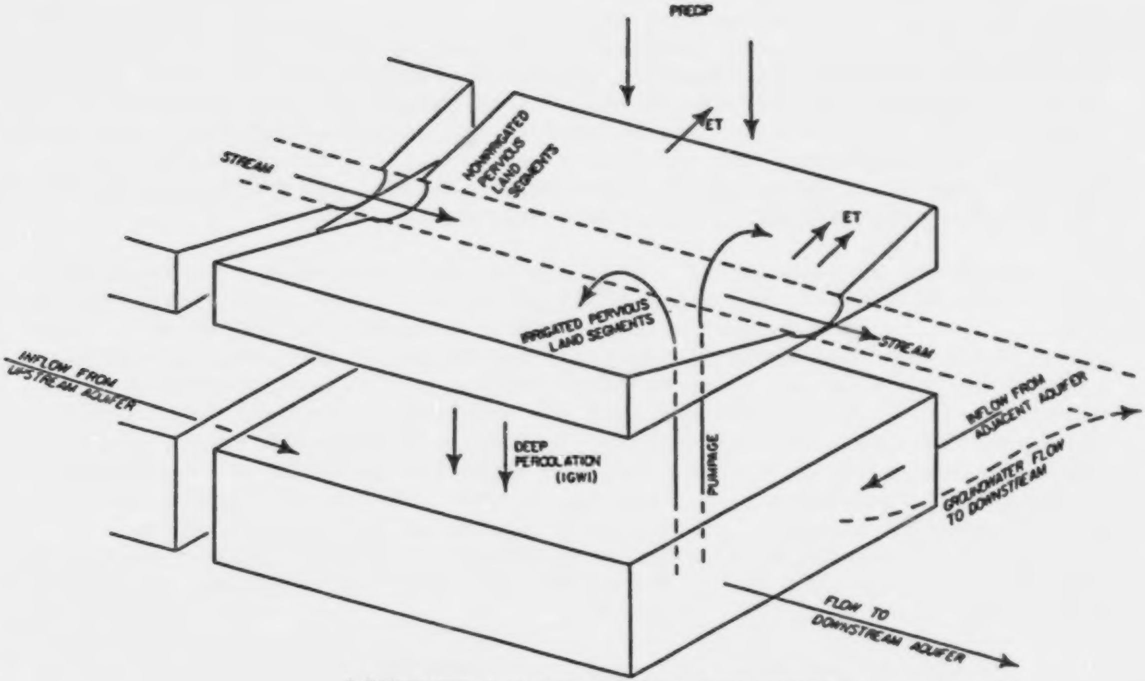


Figure 3 - Basic fluxes in HSPF simulation with groundwater pumpage.

Figure 4


PRECIP
ET
NONIRRIGATED PERVIOUS LAND SEGMENTS
STREAM
ET
IRRIGATED PERVIOUS LAND SEGMENTS
STREAM
INFLOW FROM UPSTREAM AQUIFER
DEEP PERCOLATION (IGWI)
PUMPAGE
INFLOW FROM ADJACENT AQUIFER
GROUNDWATER FLOW TO DOWNSTREAM
FLOW TO DOWNSTREAM AQUIFER

LAYOUT FOR A TYPICAL PERVIOUS LAND SEGMENT
INCORPORATED IN THE HSPF MODEL FOR THE BIG BLUE

3. Lincoln Creek at Seward: October 1953-September 1955 and
October 1964-September 1967
4. Big Blue near Crete: October 1953-September 1955 and
October 1964-September 1967
5. West Fork Big Blue: October 1958-September 1959 and
October 1964-September 1967

The first step in the calibration procedure was to match the annual flow volumes between simulated and recorded flows. Parameters related to the storage capacities of the unsaturated soil zones, evapotranspiration and infiltration are important determinants of runoff and volumes. Guidance for initial values of such parameters as lower zone moisture storage (LZSN), upper zone nominal storage (UZSN), infiltration (INFILT), lower zone evapotranspiration (LZET), interflow (INTFW), and groundwater recession (AGWRC) were taken from the suggested values in the Agricultural Runoff Model Users Manual (Donigian et al, 1978). Some guidance on parameter values was also taken from a concurrent HSPF project on the Dee Creek Watershed in southeastern Nebraska conducted by the Water Resources Research Center of the University of Nebraska (NWRC, 1982). The UZSN, INFILT, and LZET parameters were varied monthly. In calibrating parameters for annual flow volumes, adjustments were made in LZSN and INFILT.

Since LZSN and INFILT indirectly affect the actual evapotranspiration rates, decreasing LZSN and INFILT makes less water available for ET losses and increases runoff. In simulating the daily and seasonal low flow volumes, adjustments were primarily made in INFILT and AGWRC parameters. AGWRC controls groundwater outflows, while INFILT controls inflow to the groundwater. However, care needs to be taken in adjusting INFILT at this step. Since INFILT is indirectly related to LZSN, significant change of INFILT affects the long term flow volumes calibrated in the previous step. Consequently, most of the adjustment needs to be in the AGWRC term. Once the annual flow volumes and seasonal low flows are adequately calibrated, the shapes of the hydrographs were adjusted by varying the interflow inflow parameter (INTFW). Some of the salient calibration parameters for each of the stream segments simulated are summarized in Table 1.

In calibrating the model for the two periods with and without irrigation development, consideration was given to the change in evapotranspiration and interception rates from irrigated and nonirrigated lands. ET is affected considerably by the type of plant and plant density. Also, loss by interception depends on the extent of vegetal cover and density of prevailing plants and trees. It was, therefore, necessary to numerically represent the difference in monthly INTERCPT and LZETPARM between irrigated and nonirrigated conditions. The relationship between water loss and crop yield can range from linear to curvilinear response functions (ASAE, 1980). A 1:1 correlation of ET loss to crop yield was assumed based on a linear relationship of dry matter yield to relative seasonal evapotranspiration. Since corn is the major irrigated crop, the value of MON-LZETPARM and MON-INTERCEP for the months of June through September under irrigated conditions were generated by prorating the ratio of irrigated crop yield to nonirrigated crop yield. The results of a typical calibration run are illustrated in Figure 5.

VERIFICATION

Verification of the hydrologic calibration was carried out by running the models of each stream for the period October 1953 through September, 1960 for the pre-irrigation development conditions, and for the period October 1960 through September 1975 for the irrigation development conditions. Figure 6 illustrates graphically the results of typical verification runs for a simulated stream.

RESULTS OF THE CALIBRATION AND VERIFICATION PROCESS

The calibration process was particularly valuable in assigning values to prominent land parameters which were seen to be dependent upon soil type, land cover and on regional meteorological characteristics. The parameter sensitivity runs indicate that simulated flows were most sensitive to the values of LZSN, UZSN, and INFILT. In certain cases, the impacts of the monthly values of UZSN, MON-LZETPARM, and MON-INTERCEP on monthly distribution of runoff were more pronounced. In all the streams simulated, the first attempt to calibrate included periods of high runoff extending over a period of about three months, it was then extended to include longer periods of up to three years. The simulated monthly runoff volumes varied from the recorded volumes by 10 to 20 percent for the wet months and by 10 to 50 percent for the drier months. For the extended three-year calibration periods, the simulated annual runoff volumes varied from 7 to 15 percent of the recorded volumes. The correlations between measured and simulated flows were evaluated by plotting double mass curves of cumulative runoff volumes and comparing them with a line of one-to-one correspondence. A typical double mass curve of measured and simulated flows is illustrated in Figure 7. The noticeable difference between recorded and simulated annual flows resulted from relatively poor simulation of a few months in which measured flows in the drier months were lower than simulated flows. This is particularly evident during the initial months of simulation in which the parameters representing the initial conditions may be responsible for poor

TABLE 1

SUMMARY OF CALIBRATED PWATER PARAMETERS

PARAMETER	Big Blue at Surprise	Lincoln Cr. at Seward	Big Blue at Seward	West Fork at Dorchester	Big Blue at Crete
LZSN	8.0	6.0	6.0	6.0	4.0
LZS	4.0	3.5	3.5	3.5	6.0
INFILT	0.02	0.01	0.01	0.015	0.01
IRC	0.60	0.60	0.60	0.40	0.20
INFILD	2.0	2.0	2.0	2.0	2.0
AGWRC	0.99	0.999	0.999	0.999	0.99
DEEPFR	0.19	0.19	0.19	0.15	0.10
AGWETP	0.5	0.5	0.5	0.35	0.30
IFWS	0.01	0.01	0.01	0.01	0.01
SURS	0.10	0.10	0.10	0.10	0.10
UZS	4.0	4.0	4.0	4.0	4.0
UZSN*	0.13-0.32	0.13-0.29	0.13-0.30	0.05-0.15	0.19-0.29
INTERCEP*	0.03-0.30	0.03-0.30	0.03-0.30	0.03-0.20	0.01-0.30
LZETP*	0.10-0.70	0.05-0.42	0.05-0.42	0.03-0.30	0.03-0.70

* Varied monthly, only ranges shown in table.

LZSN = Lower zone nominal storage, inches
LZS = Initial lower zone storage, inches
INFILT = Mean infiltration rate index, in/hr
IRC = Interflow recession rate, per day
INFILD = Ratio of maximum to mininum infiltration rate
AGWRC = Active groundwater recession rate, per day
DEEPER = Percentage of groundwater to deep aquifer
AGWETP = Fraction of ET from active groundwater storage
IFWS = Initial interflow storage, inches
SURS = Initial surface detention storage, inches
UZS = Initial upper zone storage, inches
UZSN = Upper zone nominal storage, inches
INTERCEP = Interception storage capacity, inches
LZETP = Lower zone ET parameter

simulation. In certain cases readjustments of interflow parameters and roughness coefficients resulted in closer runoff volumes and improved timing of peaks. The difference between observed and simulated flows in verification runs was somewhat greater than in calibration runs because in calibration greater emphasis was given to higher than normal runoff events, while the verification period included both high and low periods of flows. Consequently, the calibrated parameters were somewhat biased toward high and moderate flow conditions.

The timing of snowmelt events in March and April was not well represented. This is probably due to both the use of several precipitation data series with data synthesized from other stations to substitute for missing records, and the inability of the model to accurately represent frozen ground conditions. The INFILT and UZSN parameters were reduced during the winter months, and the snow input (SNOWCF-parameter) was increased to match the volume of measured surface runoff. Additional research and model development work is needed to better understand and represent frozen ground conditions and the timing and volume of snowmelt runoff.

Some erractic behavior of the model in simulating runoff for some major runoff producing storm events is primarily attributed to the characteristics of those storm events. Review of precipitation data from several stations revealed the frequent occurrence of scattered convective storms. These convective storms

Figure 5

RECORDED AND SIMULATED HYDROGRAPHS
FOR BIG BLUE RIVER AT SEWARD

PRE-IRRIGATION DEVELOPMENT CONDITIONS (1954)

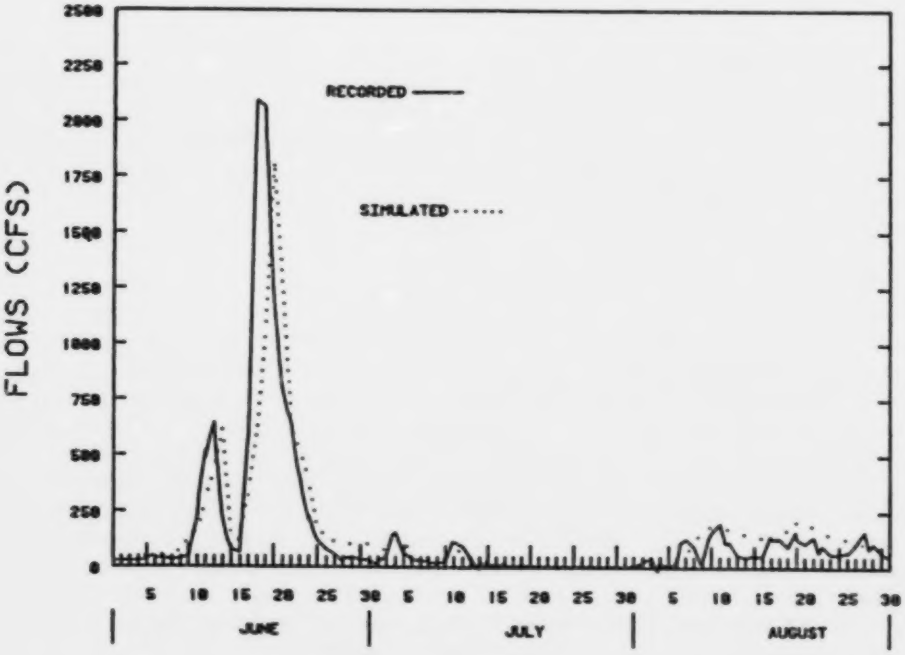
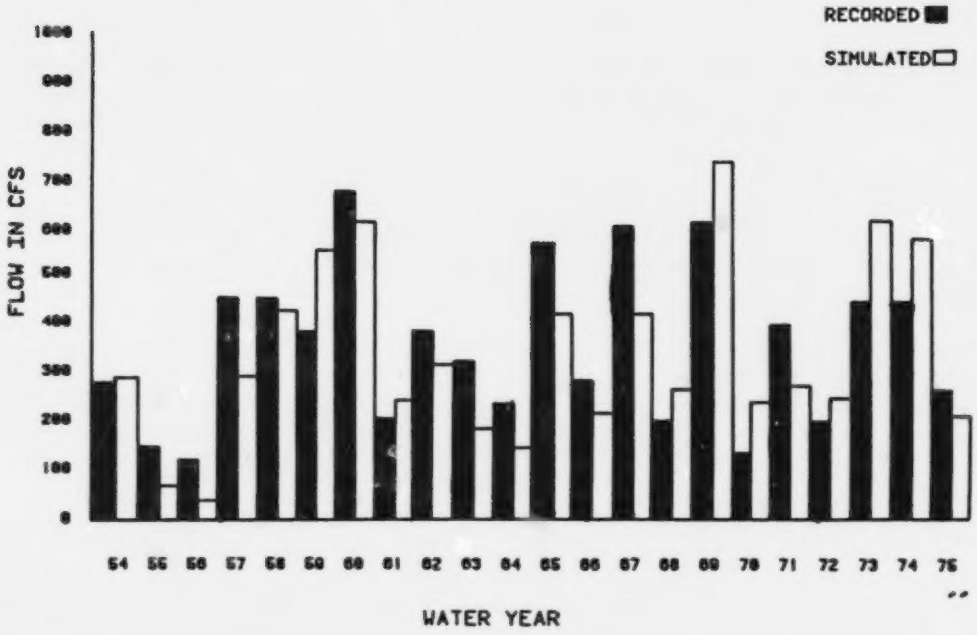


Figure 6

RECORDED AND SIMULATED ANNUAL
RUNOFF VOLUMES FOR

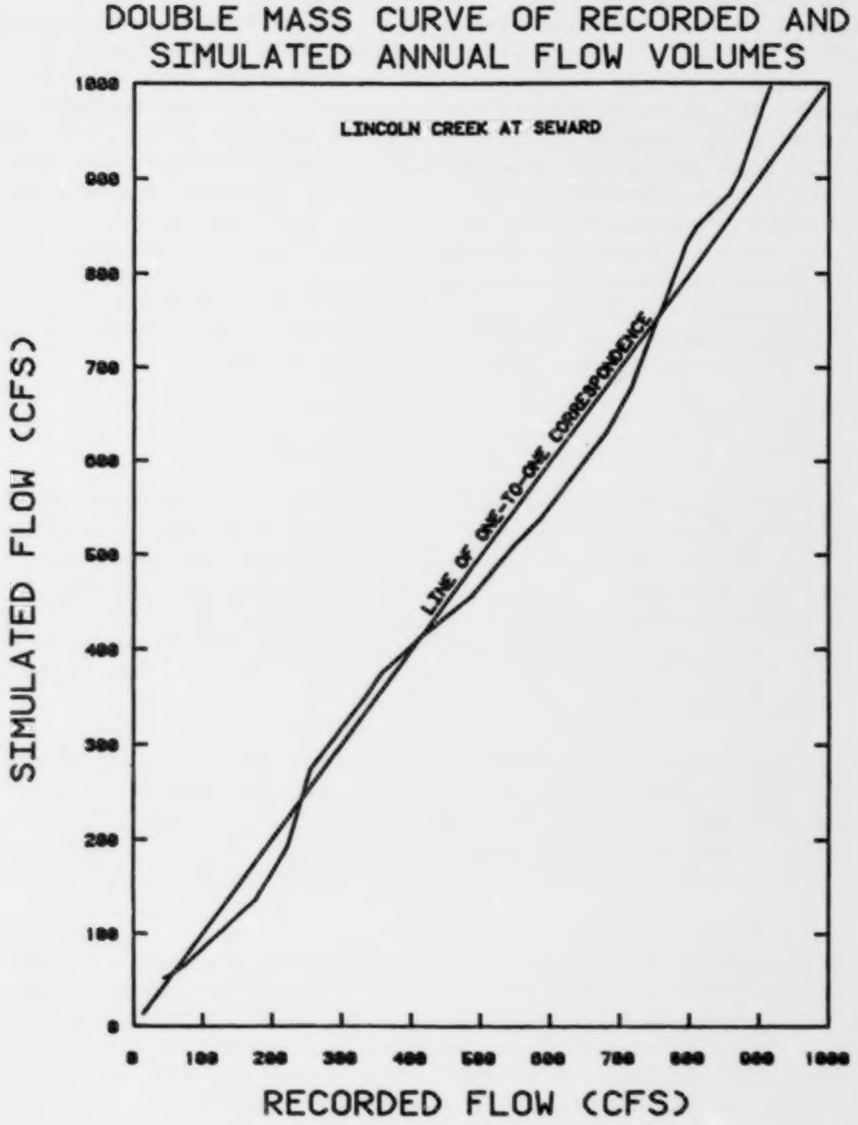
BIG BLUE RIVER AT CRETE



result in very nonuniform distribution of rainfall over a relatively small area. These types of storms, which produce localized flooding, are numerous in the Big Blue basin. For such storms, adequate representation of the rainfall-runoff relationship was not achieved because there were not enough weather stations in the model area to accurately measure the rainfall. Correcting this deficiency would require more precipitation gages in order to better represent and understand the rainfall-runoff relationship.

In simulating deep aquifer pumpage, the available algorithms of HSPF can not directly add or subtract specific volumes of water to or from a deep aquifer. This is accounted for by the introduction of the parameter DEEPFR, the fraction of groundwater lost to deep aquifer. The value of DEEPFR was arbitrarily lowered for the pre-irrigation development condition without the availability of any physically interpreted value. Secondly, the surface water-groundwater interaction in HSPF is accounted for by the use of a time series of continuous lateral outflow from active groundwater storage. A time series of active groundwater inflow AGWI was used, but the impact of external lateral groundwater inflow AGWLI, which could also have continuous infow, was not accounted for. A percentage of the pumpage rate could have been used in the time series to represent the possible increased leakage of water into the deep aquifer. Consideration should have been given to linking these values of AGWLI and DEEPFR with output from the groundwater model.

Figure 7



OBSERVATIONS

Groundwater pumping has indeed had a significant effect on the baseflow contributions to the streams in the basin. As pumping caused the water table to decline, it decreased the former gradient towards the stream which in turn decreased the discharge of the aquifer to the stream, or it reversed the water table gradient between the aquifer and the stream, which induced streamflow to seep to the underlying aquifer. These effects did not instantaneously reach the streams, but rather lagged behind the operation of pumpage depending upon aquifer properties and distance from the wellfield to the stream.

The trend observed in the simulated hydrograph series as illustrated in Figure 8 shows that the effect of irrigation pumpage was to increase surface runoff and interflow from irrigated lands initially. Actual evapotranspiration also increased. Initially, the effect of pumping is a net increase in the quantity of streamflow due to surface runoff contribution from irrigated water. This is apparent, as indicated by the early stages of the hydrographs, in almost all of the streams. As pumping continues and the aquifer water level drops, the deep percolation from the active groundwater storage that feeds streams increases and no longer contributes to streamflow. Finally, a new steady state balance is achieved where measured streamflow is less than before pumping began. For instance, in Lincoln Creek, as illustrated in Figure 8, the wasted runoff from irrigated lands increased the simulated average flows in the months of August and September in 1970 by approximately 5 cfs. However, the reduction in baseflow contribution due to aquifer level draw-down decreased the average monthly flow by 14 cfs in October and by 2 cfs in November.

CONCLUSION

An attempt was made in this project to test the applicability of HSPF in investigating the impacts on streamflow from deep groundwater pumpage and provide some informative insights and conclusions regarding modeling of surface and groundwater interactions. Certain phases of the calibration and verification process in this project, however, could not produce the desired results. Therefore, suggested improvements need to be discussed to provide guidance for future use or development of the model. Some of the specific conclusions derived from this study are:

(a) HSPF can be used with a certain degree of success to simulate stream-aquifer interaction and possible effects of deep pumping on surface water hydrology in a Nebraska river basin. The accuracy of the results, however, is somewhat limited by the modeling complication because certain algorithms used in HSPF are not specificaly designed to simulate withdrawal from groundwater by pumping.

(b) Since HSPF can not directly add or subtract specific volumes of water to or from the deep aquifer, accurate estimation of the parameters such as DEEPFR, IGWI, AGWI, and AGWS, that play dominant roles in accounting for storage and movement of groundwater, is very important in successful simulation of groundwater pumpage impacts. If possible, the values of such parameters should be estimated from, or determined by, a groundwater model.

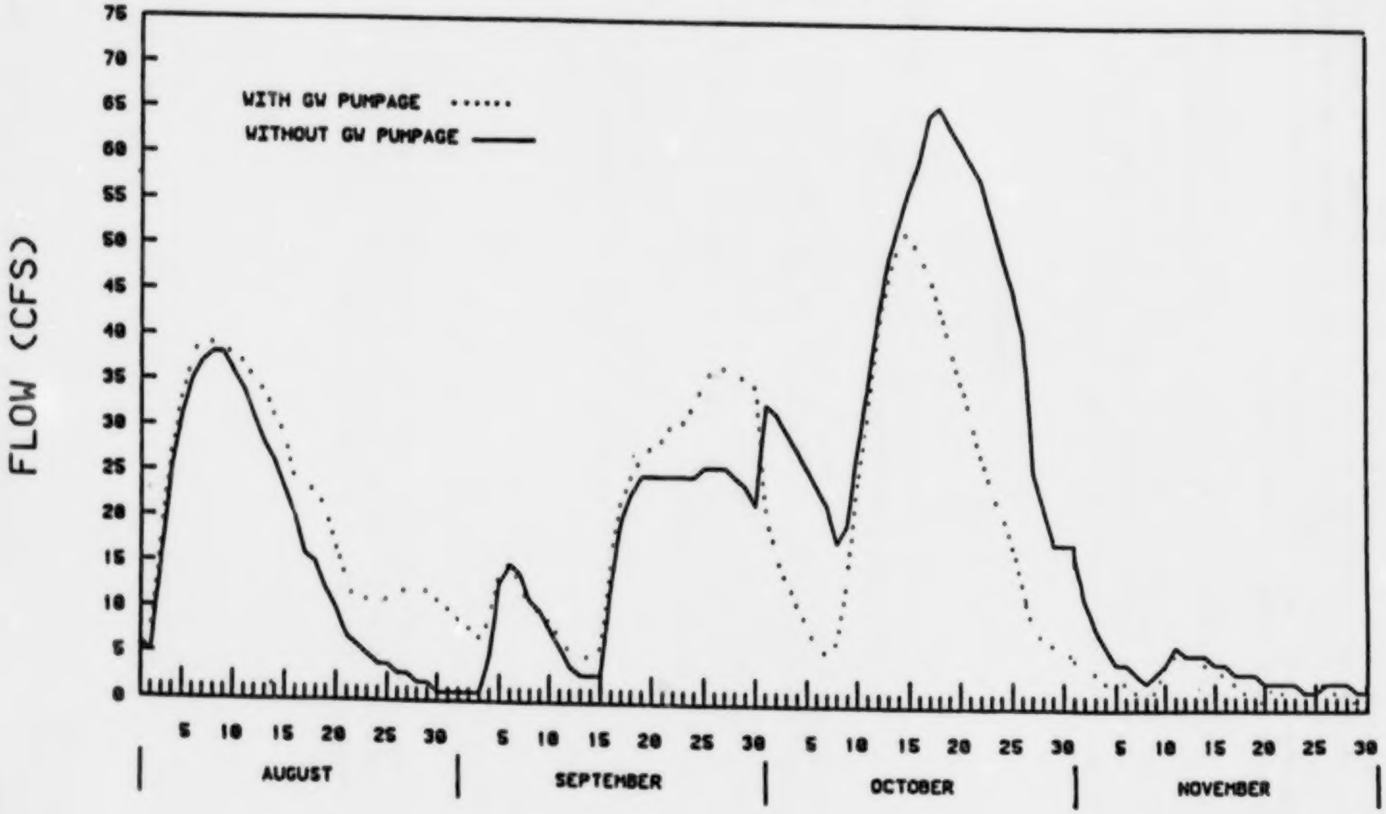
(c) Meteorological time series data, particularly precipitation, is critical to effective continuous hydrologic simulation. A denser network of precipitation records is necessary for accurate representation of the hydrometeorologic condition of a basin. Lacking this data, it is better to compare duration and frequency information from the overall simulation rather than data on an individual storm event.

(d) Equally important is the adjustment of infiltration parameters during winter and early spring to represent frozen ground conditions for simulation of

Figure 8

CONTINUAL EFFECT OF GROUNDWATER PUMPAGE
ON STREAMFLOW 1970

LINCOLN CREEK NEAR SEWARD



snowmelt runoff, and during summer to represent the degree of antecedent saturation condition for simulation of runoff from intense storm events. However, it was observed that while trying to adjust the parameters for reconstitution of high runoff events, some parameters become biased towards high and moderate flows, and do not respond to low flow events. Care should be taken in adjusting such sensitive parameters.

(e) Effectiveness of continuous process hydrologic-hydraulic modeling is affected by the size of the hydrologic land segment simulated. Due to the large size of the Big Blue Basin, the hydrologic land segments had to be relatively large in order to reduce computer costs. It was, therefore, felt that HSPF would be more effectively used to model a medium-sized watershed.

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THE USE OF SWMM TO PREDICT RUNOFF FROM NATURAL WATERSHEDS IN FLORIDA

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ABSTRACT

A large Florida watershed, the Deer Prairie Slough basin, was selected to test the ability of the EPA Storm Water Management Model, Version III, (SWMM III), to predict runoff from natural and undeveloped watersheds. The Deer Prairie Slough watershed was chosen for calibration because it has a USGS maintained stream gage, nearby National Weather Service raingage, and nearby USGS monitored wells. Also, the site is near to a GeoScience project in Sarasota County which is similar in size.

The model was calibrated to the measured runoff for the test area using site specific hydrologic data, groundwater conditions at the time of the storm event, and hourly rainfall data. A second recorded storm event was selected to verify SWMM's ability to predict measured runoff once calibrated. The model configuration for the calibration simulation was left unchanged in the verification procedure with the exception of new hourly rainfall values and new antecedent moisture conditions.

The modeling results show SWMM III to be quite accurate in predicting both peak runoff rates and runoff volumes. The relatively simple modeling strategy combined with SWMM's powerful runoff, storage, and routing features indicate the model to be a very effective tool in predicting runoff rates and volumes in Florida.

INTRODUCTION

The US Environmental Protection Agency (EPA) Storm Water Management Model (SWMM) is a comprehensive, physically based mathematical model designed to evaluate stormwater quantity and quality. The model has been widely used in the United States, Canada, and Western Europe. It is supported by the USEPA, and is updated and maintained at the University of Florida, Department of Environmental Engineering Sciences.

The principal use of the model has been to evaluate stormwater runoff from urban sewered systems. However, the rainfall, infiltration, evaporation, overland flow, channel routing, and detention storage algorithms used in SWMM are not peculiar to urban sewered catchments, but are basic to any stormwater analysis. These features, in fact, make SWMM well suited for the evaluation of before and after development runoff peak flows and volumes as required by development permitting regulations of the State of Florida. Flow attenuation by detention storage and channel routing through second and third order networks are two particularly strong features of the SWMM TRANSPORT module.

The purpose of this study was to determine SWMM's modeling accuracy on pre-development sites in Florida. Of particular interest was the model's ability to simulate large watersheds as is commonly required in large development projects and regional or county-wide drainage studies.

SITE AND DATA DESCRIPTION

For this work, a large undeveloped watershed was selected which has a history of recorded stream gage data monitored by the USGS. This basin is Deer Prairie Slough located in the southern portion of Manatee County and the northern portion of Sarasota County as shown in Figures 1 and 2. The watershed is composed of 27,016 acres of flat terrain interspersed with natural depressions and wetland areas. Shallow groundwater conditions and sandy soil types are characteristic of the area.

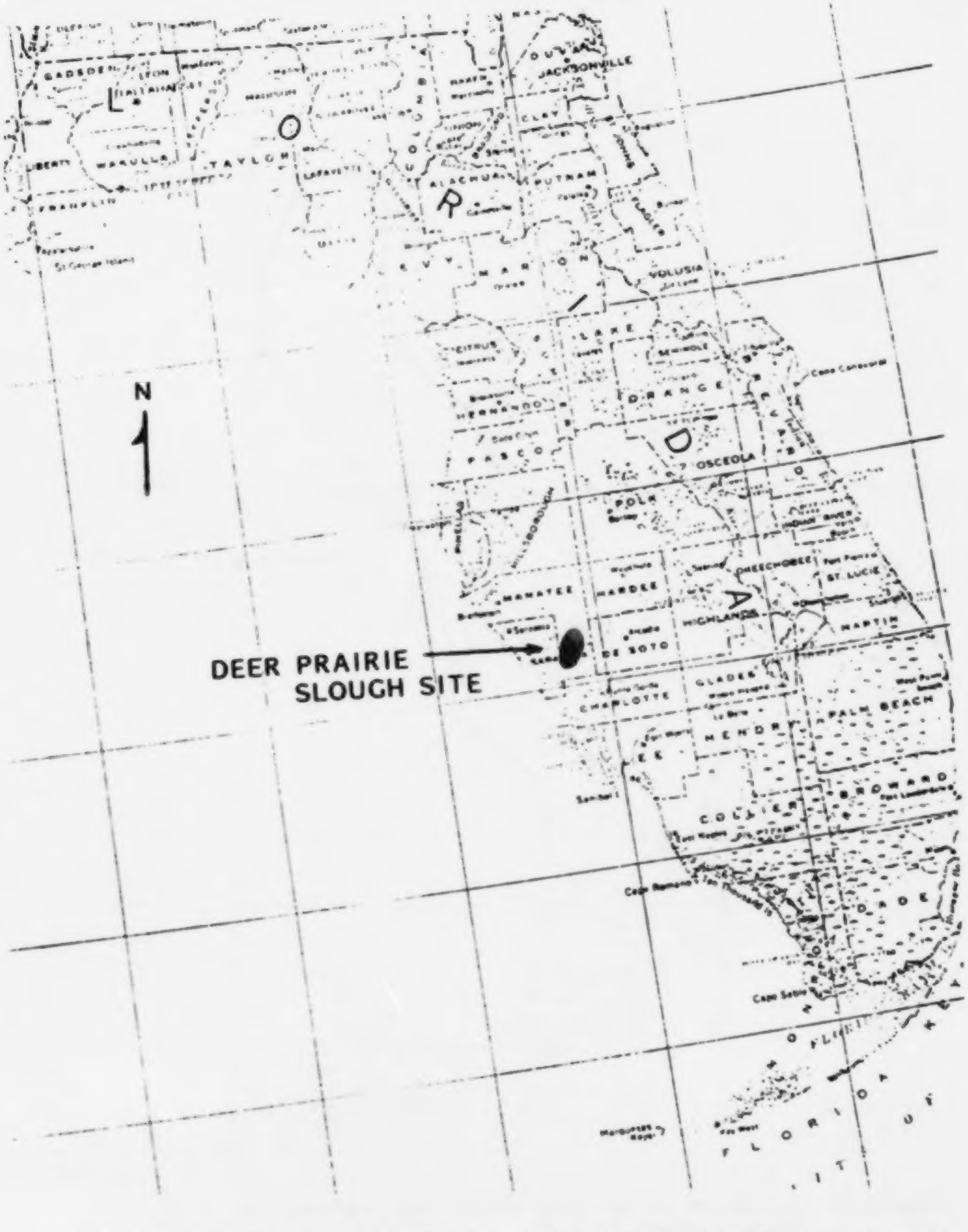
Rainfall data were taken from the Venice, Florida station, within approximately 10 miles of the site. The data are recorded in hourly increments beginning in 1942 (NWS).

From this rain station a single storm event recorded June 17 and 18, 1982, was selected for calibration to the site. This 8.0 inch event over a 24-hour duration approximates the design storm required by Sarasota county of a 9.5 inch, 24-hour event.

A storm event recorded September 12 through 26, 1982 was chosen to verify the modeling results. This event occurred over 109 hours and contained a depth of 6.6 inches (NWS). The major portion of the rainfall took place over the first 26 hours.

Groundwater conditions were simulated from data taken from the Big Slough Shallow Well, ROMP 19ES, and ROMP 19WS shallow USGS monitoring wells near the site (USGS, 1982b). The well locations are shown in Figure 2.

Data for the June 17 and 18, 1982, storm event used for the calibration of the site, showed groundwater conditions at an average depth of 2.46 feet below surface as recorded on June 15. For the verification trial, groundwater data measured on September 20, 1982, indicated an average depth of 1.96 feet below surface.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ORANGE
PASCO
OSCEOLA
POLK
MANATEE
HARDEE
HIGHLANDS
OKEECHOBEE
ST. LUCIE
MARTIN
DE SOTO
GLADES
CHARLOTTE
PALM BEACH
HENDRY
BROWARD
COLLIER
DADE

Figure 1. Location of the Deer Prairie Slough Watershed in Florida.

METHODOLOGY

Proper simulation of runoff events in much of Florida requires an understanding of the effect of the ground water table on surface flow peaks and volumes. Large storms, such as those approximating so called "design storms", generate overland flow by filling the void space in the soil until the water table rises to the surface. The hydraulic conductivities of Florida soils in

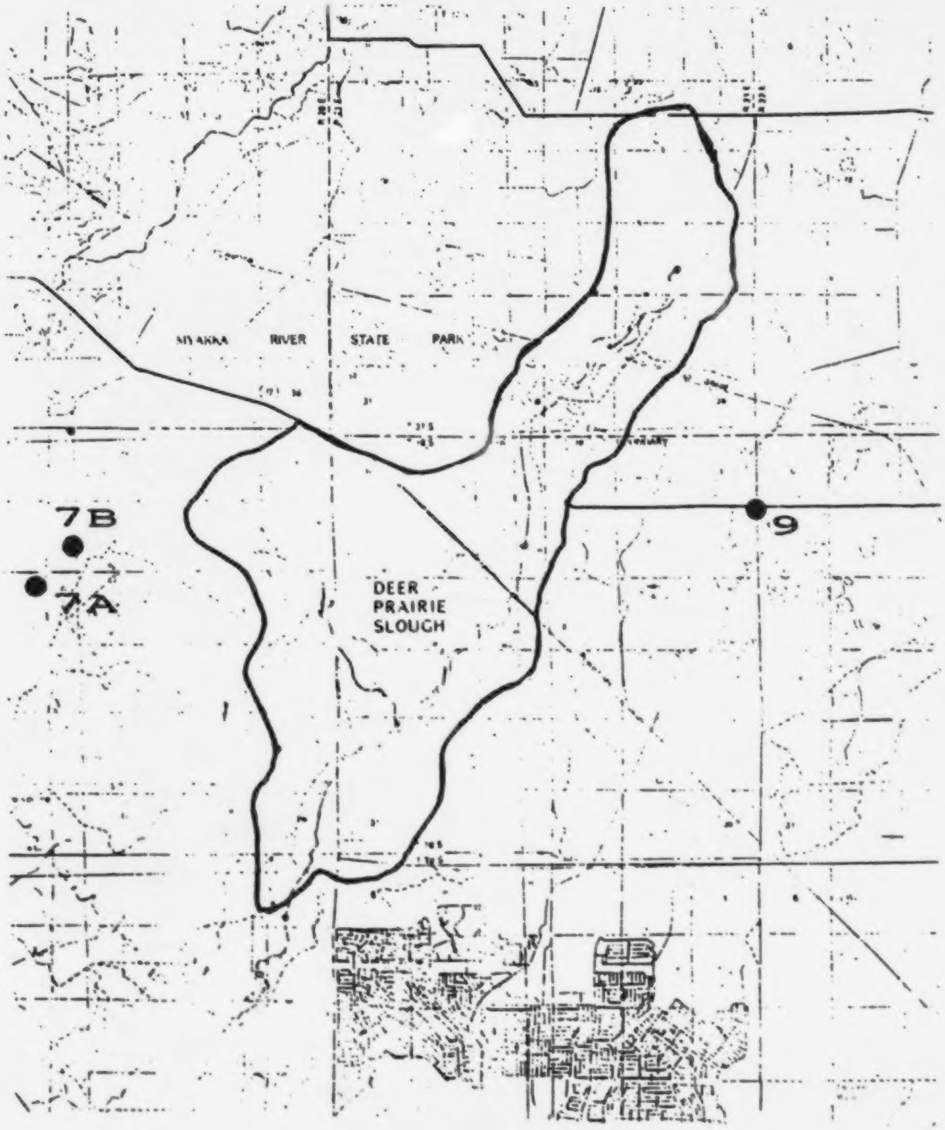

Figure 2. Location of Shallow Wells with respect to Deer Prairie Slough.

these high water table areas are greater than the peak intensities of even design sized storms. Thus, runoff is generated by excess rainfall after the water table has risen to the surface, not by high intensity rainfall on soils of low hydraulic conductivity.

This condition may be simulated by modeling each sub-basin as a shallow reservoir which must be filled before outflow, expressed as overland flow, can occur. The simulation is done in the SWMM RUNOFF module by setting the Horton or Green-Ampt infiltration parameters to low values, essentially turning them off. Depression storage over the area is then employed as the reservoir which must be filled before overland flow can occur.

Determination of the proper amount of depression storage is a function of depth to water table and water storage capacity above the water table. Depth to water table information may be obtained from the US Geological Survey Water Resources Data Publications, the Water Management Districts or local data sources. Soil moisture storage information is available in publications such as Characterization Data for Selected Florida Soils available from the Institute of Food and Agricultural Sciences at the University of Florida (Carlisle, et al., 1981). The South Florida Water Management District (SFWMD) recommends the following typical values for storage above the water table in most South Florida soils (SFWMD, 1984):

Depth of Water Table (ft)	Storage (in)
1.0	0.6
2.0	2.5
3.0	6.6
4.0	10.9

These SFWMD values were plotted to aid in interpolation of soil moisture storage values (Figure 3).

The watershed was divided into six sub-catchments along topographic features and flow patterns. Figure 4 indicates the sub-basin delineations. Sub-catchment widths were determined for model input by taking cross section widths perpendicular to the direction of flow at the sub-catchment boundary. Areas, widths and slopes were taken directly from topographic maps. Depth to water table was determined by averaging values from wells near or in the watershed, and water storage values were determined from the SFWMD curve. These values were input as pervious depression storage. Wetland areas were planimetered and input as containing 1.0 inch of depression storage within the wet season hydroperiod elevations.

Flows were routed using the SWMM TRANSPORT module. Channels were simulated as broad trapezoidal channels having side slopes typical of the topography along the natural channels. A Manning's roughness value of 0.35 was used for overland flow to simulate undeveloped conditions (Huber et al., 1981). Channel lengths and bottom slopes were taken from the topographic maps. The watershed outflow hydrograph was plotted at one-half hour intervals. On the same hydrograph for comparison purposes were plotted the measured data from the USGS gauge.

DISCUSSION

The 8.0 inch, 24 hour storm of 17 June 1982 was used in the calibration of the outflow from Deer Prairie Slough. The only calibration parameter used to adjust the predicted outflow was the sub-catchment width. The final values used were similar to measured topographic values. The peak outflow for the storm was the largest of the year for the gage and was recorded as 387 cfs, instantaneous (USGS, 1982a). The simulated peak was also calibrated to 387 cfs. Table 1 lists pertinent data from this calibration

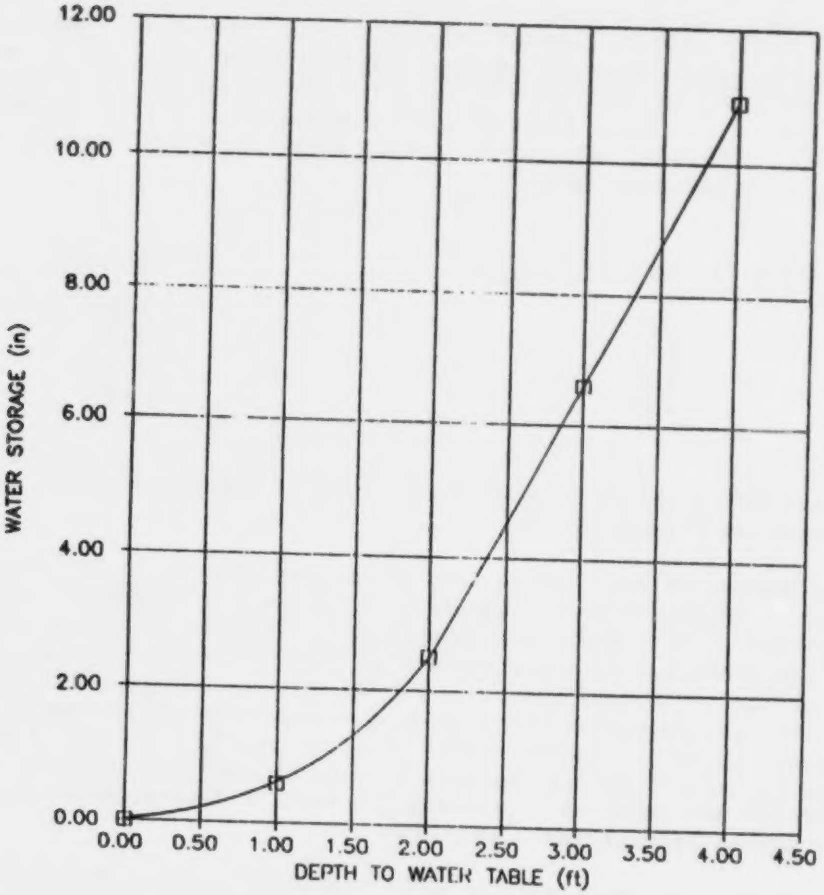

Figure 3. Soil Moisture Storage as a Function of Depth to Water Table (as per SFWMD, 1984).

simulation. The volume calibrations performed within the model indicated a measured volume of 2.85 inches compared to a predicted volume of 2.51 inches. The measured and predicted hydrographs are shown on Figure 5.

Table 1: DEER PRAIRIE SLOUGH CALIBRATION DATA

WATERSHED SIZE:	27,060	acres
AVERAGE DEPTH TO WATER TABLE:	2.46	feet
TOTAL PRECIPITATION:	8.0	inches
STORM DURATION:	24	hours
PREDICTED PEAK:	387	cfs
MEASURED PEAK:	387	cfs
PREDICTED VOLUME:	2.51	inches
MEASURED VOLUME:	2.85	inches

The second Deer Prairie Slough storm occurred 21-25 September 1982, when 6.6 inches of rain fell at the Venice raingage (NWS). No watershed parameters were adjusted for this simulation. The only changes from the calibration trial were the hourly rainfall values for the storm and the moisture storage capacity above the water table. The measured and predicted outflow hydrographs are shown in Figure 6. Features of the two hydrographs are similar to those of the first storm. The predicted outflow of 264 cfs is 1.8% greater than the measured 260 cfs which is a daily average (USGS, 1982a). The predicted volume is 9.4% less than the measured. Table 2 includes the data and results.

Table 2: DEER PRAIRIE SLOUGH VERIFICATION DATA

WATERSHED SIZE:	27,060	acres
AVERAGE DEPTH TO WATER TABLE:	1.98	feet
TOTAL PRECIPITATION:	6.6	inches
STORM DURATION:	109	hours
PREDICTED PEAK:	264	cfs
MEASURED PEAK:	260	cfs
PREDICTED VOLUME:	1.74	inches
MEASURED VOLUME:	1.92	inches

There are several noteworthy features of the hydrographs shown on Figures 5 and 6. The shapes of both measured and predicted hydrographs correspond very well to one another. The hydrographs in Figure 5 reflect runoff from rainfall of varied intensity over 109 hours. SWMM allows a maximum of 200 rainfall values to be input for a single event simulation, so the measured runoff "plateau" at about 260 hours which resulted from 2.90 inches of rainfall at that time is not reflected in the predicted hydrograph.

The lag noticed in both predicted hydrographs when compared to the measured hydrographs is probably due to more immediate flow resulting from the water table rising above the channel bed. The simulation generated flow when the water table rose to the land surface.

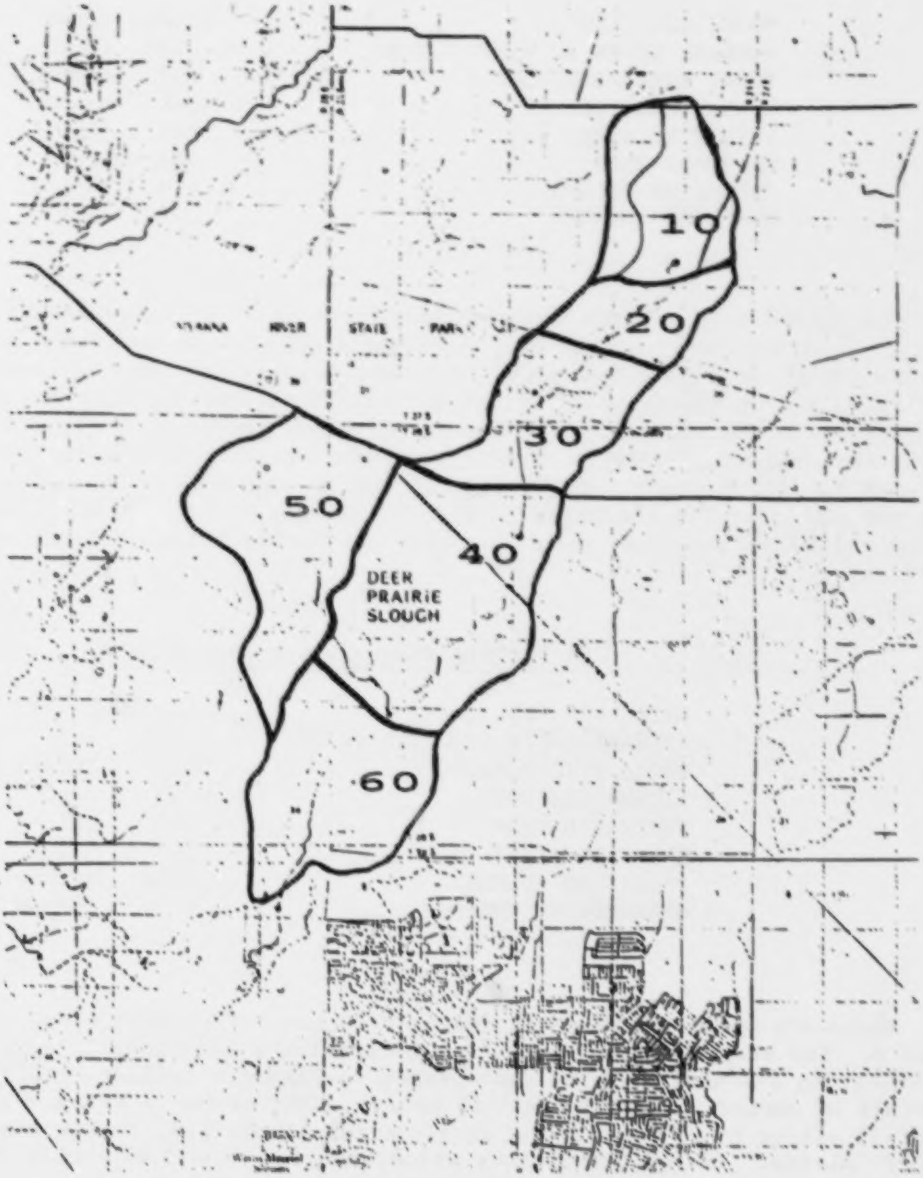
10
20
30
40
50
60
MYAKKA RIVER STATE PARK
DEER PRAIRIE SLOUGH

Figure 4. Sub-basin Delineations within Deer Prairie Slough.

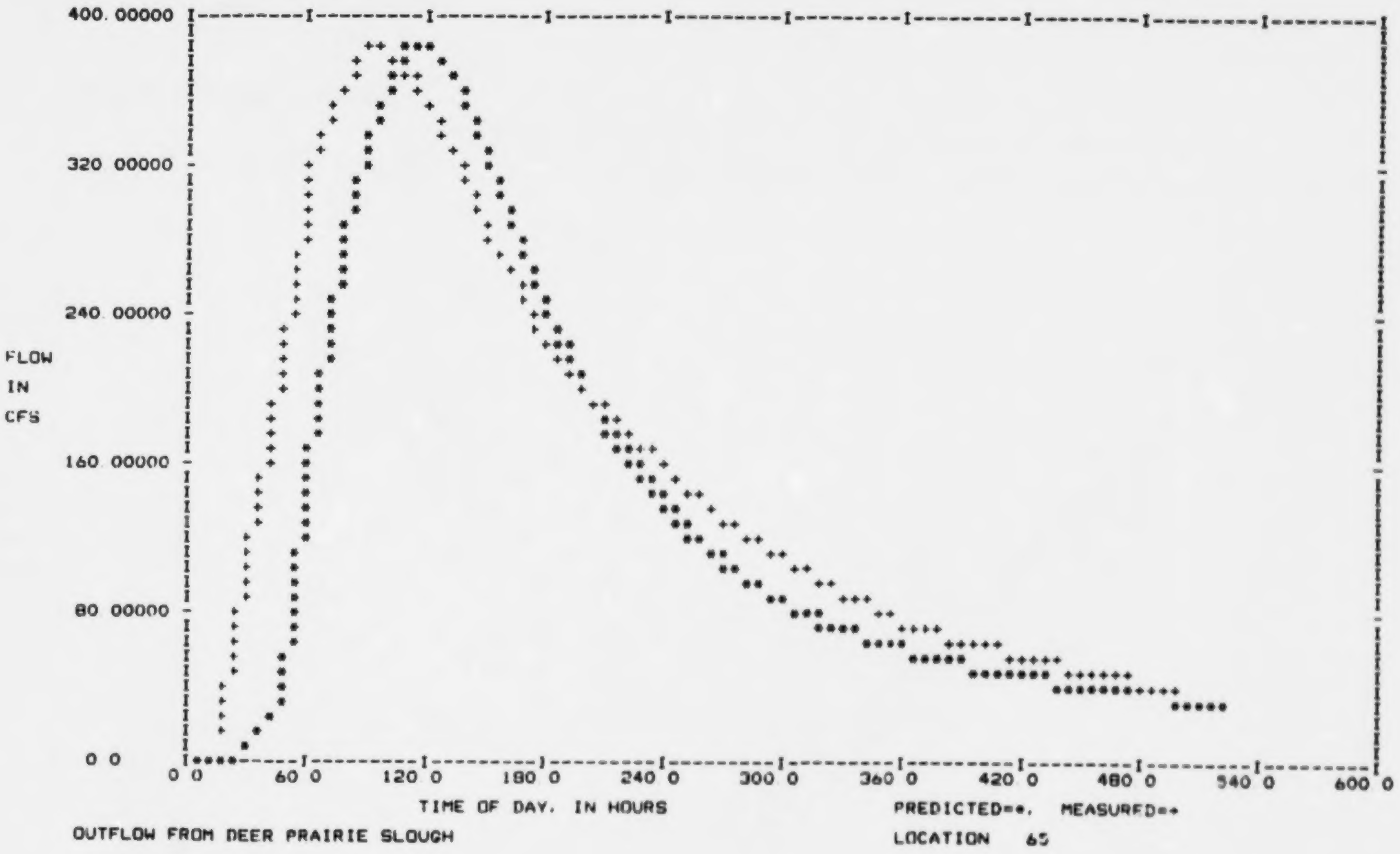

Figure 5. Measured and Predicted Hydrographs for the Storm of 17 June 1982.

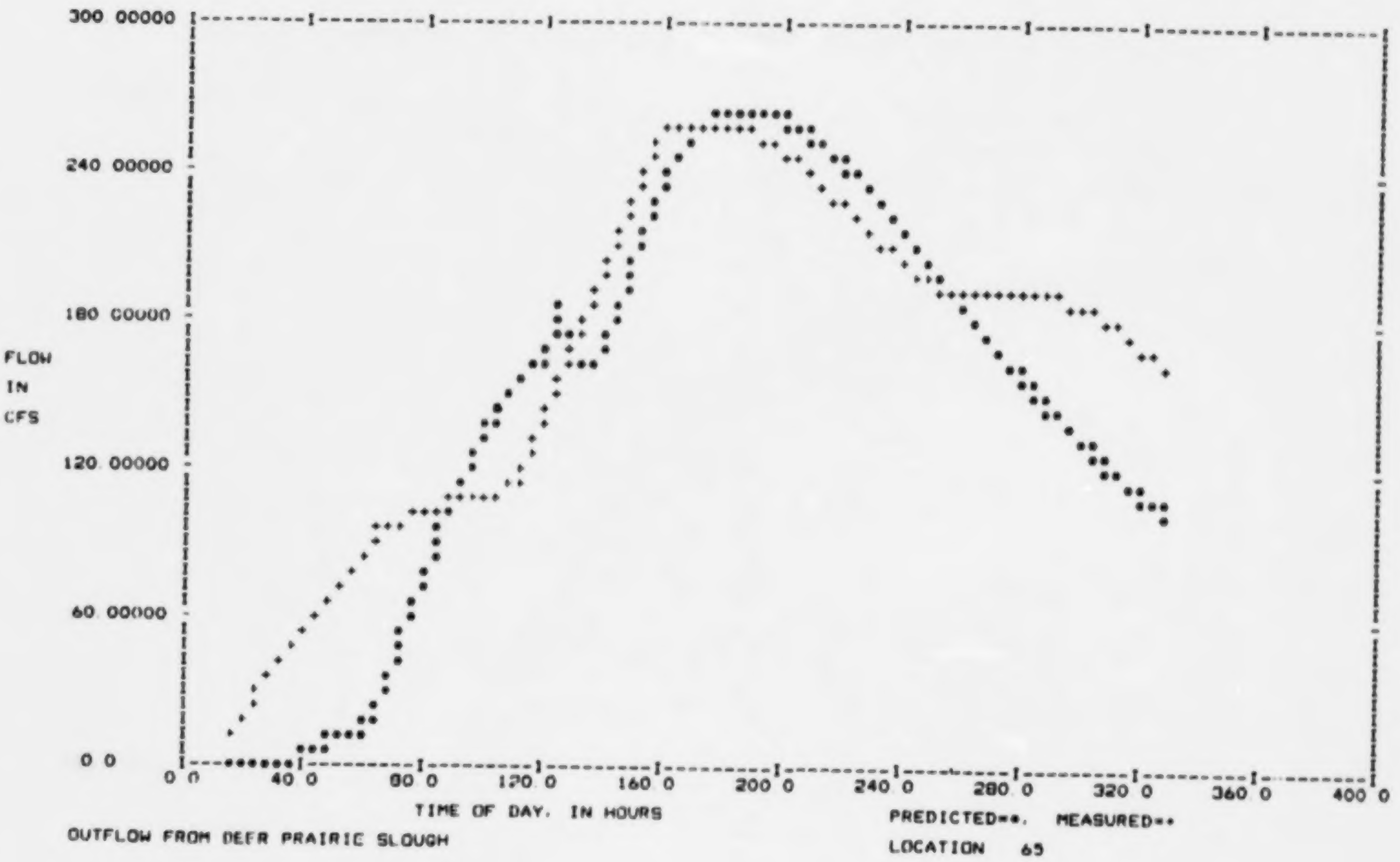

Figure 6. Measured and Predicted Hydrographs for the Storm of 21-25 September, 1982.

CONCLUSION

The Storm Water Management Model proved to produce good results when compared with measured data for the site. The match of peak flows generated by the storm events was in very good agreement with the measured peaks while the volumes predicted by the model also compared favorably with the actual data.

Little initial calibration was required to match predicted to measured peak discharge values, with the width parameters being the only input value that was adjusted. SWMM is able to predict runoff events for very large watersheds, which gives it a distinct advantage over other modeling methods such as the SCS and Rational Methods.

One of the more important factors for accurately modeling the events appears to be the simulation of the water table depth prior to the storm. Information about the water table depths is necessary in order to model the antecedent moisture conditions which are critical to Florida runoff from large storms.

It is also important for larger sites to be broken down into sub-catchments. This allows the TRANSPORT module to account for flow routing from one sub-catchment to another rather than producing one output hydrograph based upon homogeneity within the watershed.

The use of SWMM allows physically measurable, site specific input to be modeled rather than parameters derived from regression analysis of regional data. The model also allows definition of worst case conditions as well as wet season averages.

SWMM's capability to include site-specific data and antecedent moisture conditions together with its channel network routing and detention storage capabilities make it an excellent choice for accurately predicting runoff peak discharges and volumes on high water table Florida watersheds.

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6. U.S. Geological Survey Water-Data Report, 1982(b), Water Resource Data - Florida, Water Year 1982. Vol. 3B. Southwest Florida Ground Water, FL-82-3B.

The work described in this paper was not funded by the U.S. Environmental Protection Agency and therefore does not necessarily reflect the views of the Agency and no official endorsement should be inferred.

A SIMPLIFIED WATER QUALITY COMPUTER PROGRAM FOR REGIONAL STORMWATER MANAGEMENT SITE EVALUATION

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ABSTRACT

Montgomery County, Maryland, has recognized the advantages of regional stormwater management facilities since the late 1970s. Until the last few years, the design of SWM facilities was based on reducing the peak discharge for the post-development conditions to the corresponding peaks for pre-development conditions for the 2-, 10-, and often the 100-year storms. It was known that SWM facilities also provide qualitative benefits. No compatible method was available to quantify these benefits that would require an effort similar to the work needed to compute the peak reduction.

The desire to produce a simple method to evaluate the water quality benefits of SWM facilities and recent legislation concerning watershed management policy in the State of Maryland have led Montgomery County and their engineer, Greenhorne & O'Mara, Inc., to develop a simple computer program to evaluate the water quality impacts of SWM ponds throughout a watershed. The model was developed to utilize data prepared for USDA Soil Conservation Service Technical Release Number 20 because TR-20 is frequently applied throughout the State for quantitative design of SWM facilities.

A rainfall amount-frequency relationship was developed from data collected by the County. Pollutant accumulation, pollutant washoff, and pond trap efficiency information were obtained from previous works.

The computer model developed was applied during two watershed studies conducted for the County by Greenhorne & O'Mara. The output from the model aided in the evaluation of alternate SWM facility sites located throughout the watershed. The model enabled a comparison of pollutant levels at various points along the main stem and the larger tributaries similar to a comparison of the peak discharges that had been accomplished for previous studies. Since this water quality analysis required little additional data, the new analysis was accomplished for a minimal cost.

INTRODUCTION

Regional stormwater management facilities have been designed and constructed in Montgomery County, Maryland, since the late 1970s. The equations that govern the quantity of runoff that is stored and released have long been known. Methodologies that utilize the storage and outflow relationships have been developed into small dam and stormwater management pond design procedures.

During the earlier years of the County stormwater management program, the emphasis had been placed on pond release rates, downstream channel velocities, and other hydraulic parameters. However, more recent concerns about the levels of sediment and nutrients being carried to the Patuxent River, the Chesapeake Bay, and other valuable bodies of water have modified the rationale for the selection of regional stormwater management sites towards sites that improve the quality of the runoff.

Water quality models have been in use for more than fifteen years. The models, such as STORM, SWMM, NPS, WASP, HSP, SEDIMENT, and DEPOSITS, were developed for various levels of planning and design applications. A model was needed for which data could be easily developed and would be compatible with the water quantity model most frequently used in the State of Maryland for development impact studies, the USDA Soil Conservation Service TR-20 computer program (1).

RAINFALL AMOUNT-FREQUENCY

DATA ANALYSIS

Montgomery County has an on-going rainfall data collection program at sites located throughout the County. The length of record for each site is approximately the same. A site near the center of the County was selected to be representative of the precipitation received County-wide.

Daily precipitation data were available from January 1980 through December 1983. Daily rainfalls greater than one-half inch were considered significant for pollutant washoff. A tabulation of rainfall amounts exceeding this value was compiled, and the rainfall amounts were plotted against frequency.

Rainfall amounts for selected frequencies were read from the plot and are presented in Table 1. Although a limited number of years of data was available, the results obtained were judged to be reasonable because a large number of the very frequent storms occurred during the period of record. In addition, the data plotted well compared to rainfall amounts obtained from Technical Paper 40 for 1 year and 2-year return periods (2).

TABLE 1. RAINFALL AMOUNTS FOR SELECTED RETURN PERIODS IN MONTGOMERY COUNTY, MARYLAND

Frequency (months)	Rainfall Amount (inches)
24	3.2*
12	2.6*
6	1.8
3	1.2
2	0.95
1	0.6

*From TP-40 (2)

ANNUAL STORM DISTRIBUTION

In an average year, the following storms would be observed:

1. one storm greater than or equal to the twelve-month storm;
2. two storms greater than or equal to the six-month storm;
3. four storms greater than or equal to the three-month storm;
4. six storms greater than or equal to the two-month storm;
5. twelve storms greater than or equal to the one month storm; and
6. a larger number of storms less than the one month storm.

The rainfall amount associated with storms greater than or equal to the one month storm was calculated to be 12.3 inches based on the above information. Montgomery County receives an average annual rainfall of about 39 inches. The difference between the rainfall values represents a very large number of events that produce less than 0.6 inch of rain over a twenty-four hour period. These storms were not considered further because they produce insufficient runoff to wash significant amounts of pollutants downstream.

Rainfall in the County is spaced almost evenly throughout the year. Large storms may and have occurred in every month. The storm distribution developed assumed that the twelve storms greater than or equal to the one month storm were equally spaced. Therefore, one storm occurred every 30.4 days. The twelve-month storm and the six-month storm were offset by six months. The two remaining three-month storms were also placed six months apart and were offset from the twelve-month storm by three months. In this manner, the storms were distributed evenly over the twelve-month period.

POLLUTANT BUILDUP AND WASHOFF

ACCUMULATION RATES

Daily, dry weather pollutant accumulation rates for pervious and impervious fractions of urban and rural land uses were developed by the Northern Virginia Planning District Commission (NVPDC) for the Metropolitan Washington Council of Governments (COG) in 1979 (3). The NVPDC accumulation rates were judged to be acceptable for Montgomery County because of similar physiography and the proximity of the NVPDC study area and Montgomery County.

WASHOFF

The washoff function of Metcalf & Eddy, Inc., et al. was utilized as the washoff mechanism (4). The limiting pollutant load value was found to occur over a period of time shorter than the 30.4 days between each storm of significance. Hence, the maximum accumulation of pollutants was assumed to be obtained before the beginning of each runoff event.

Runoff in the washoff process was determined for a given soil type and land use through the runoff curve number (RCN) methodology of the USDA Soil Conservation Service (5). The RCN methodology was selected because the RCN values were be available from the data prepared for the hydrologic portion of the study.

POND TRAP EFFICIENCY

The principal reason for the development of the computer model was to estimate the removal of pollutants by a regional SWM facility, which can be related to the trap efficiency of the pond. In 1975, Chen developed a series of curves of trap efficiency versus the ratio of basin area to outflow rate (6). Chen's work was based on previous studies by Camp. The curves reflect the size of the sediment and the settling velocity of the sediment particles.

Settling velocity data for soils typically found in the metropolitan Washington area were presented in the NVPDC study (3). The pond retention time and the settling velocity are used to calculate the percent of each particle size which settles in the pond. The computer model developed employs a default soil gradation that reflects the general soil of the County. A different gradation curve may be input by the user.

NVPDC (3) presents a table that assigns fractions of the suspended portion of each pollutant included in the study to sediment particle sizes. The pollutants were assumed to settle as their associated sediment particles settled. The pollutants remaining in the discharge from the pond was assumed to be the dissolved pollutant plus the portion of the suspended pollutant associated with the sediment that failed to settle in the pond.

MODEL OPERATION

Execution within the model occurs in the following manner:

1. The total accumulation of sediment, BOD, total phosphorus, total nitrogen, extractable lead, and extractable zinc on each subarea prior to each storm is generated from impervious and pervious fractions for each land use;

2. The amount of washoff from each subarea for each storm is calculated based on Metcalf & Eddy's washoff function;

3. The pollutant washoff data are annualized by summing the data for the number of occurances of each storm frequency;

4. The pollutant washoff from each subarea is added based on series and parallel subarea relationships from upstream to downstream until a pond is encountered;

5. The pond routine is called, and the amount of each pollutant deposited in the pond is calculated. Basin area and outflow rates were input based on the SCS TR-20 printout for the storm frequencies studied; and

6. The subarea addition and pond routines continue until the study outfall is reached.

MODEL APPLICATION

The simple computer model - titled "WATQUAL" due to the lack of a more imaginative name - was applied in two watershed studies in the County. The results for a portion of one of the studies, the Little Paint Branch study, are included in this section.

A portion of the Little Paint Branch watershed is shown in Figure 1. The anticipated urbanization of the watershed is reflected in the increase in curve number from existing conditions to ultimate development shown on the figure.

The data input reflects combinations of soil type and land use for each subarea. These data, which had been digitized for the TR-20 model, became the bulk of the input data required for WATQUAL.

Without the proposed pond in Subarea 7, the annual sediment load at Subarea 5 would increase from 26 tons to 58 tons from existing to ultimate conditions. With the proposed pond, the sediment load would be reduced to 11 tons as shown on Table 2.

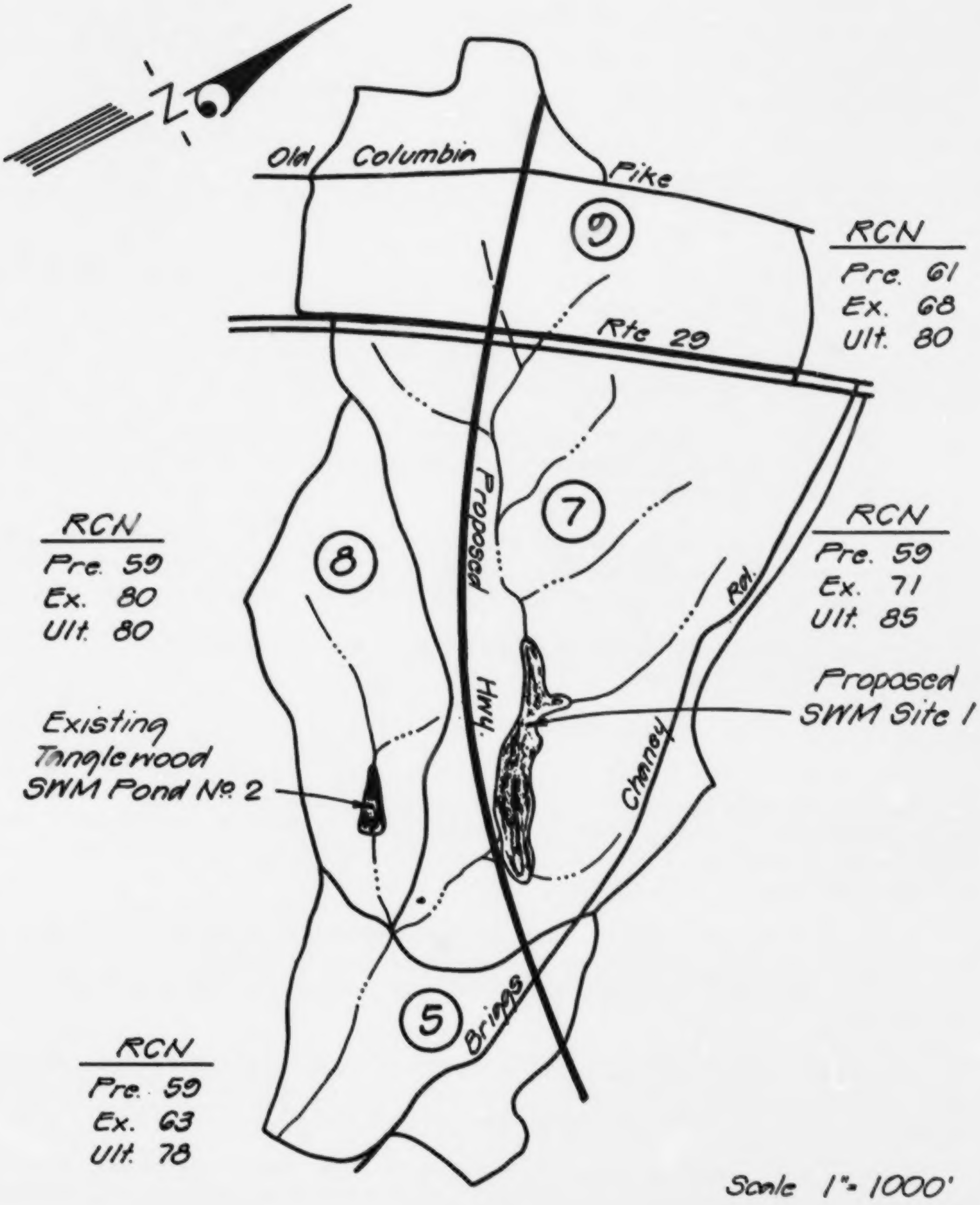
N
Old Columbia Pike
9
RCN
Pre. 61
Ex. 68
Ult. 80
Rte 29
Proposed Hwy.
7
RCN
Pre. 59
Ex. 71
Ult. 85
8
RCN
Pre. 59
Ex. 80
Ult. 80
Rd.
Proposed
SWM Site 1
Existing
Tanglewood
SWM Pond No. 2
Chaney
5
Briggs
RCN
Pre. 59
Ex. 63
Ult. 78
Scale 1" = 1000'

Figure 1. Little Paint Branch Study Example

TABLE 2. EXAMPLE STUDY SEDIMENT LOADS

Land Use Condition	Annual Sediment in Runoff (tons)			
	Subarea 9	Subarea 7	Subarea 8	Subarea 5
Predeveloped	0.39	1.04	0.20	1.42
Existing	6.97	24.09	0.92	26.23
Ultimate Without Site 1	13.73	47.92	1.06	58.33
Ultimate With Site 1	13.73	0.34	1.06	10.76

MODEL LISTING

The length of the listing of the Fortran computer model prohibited the publication of the listing with this paper. Those interested in obtaining a listing may contact one of the authors.

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The work described in the paper was not funded by the U.S. Environmental Protection Agency and therefore the contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

DEVELOPMENT OF THE HAZPRED MODEL

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[2] Environment Canada, Toronto, Ontario

ABSTRACT

HAPZRED is an interactive, microcomputer based model capable of the prediction of hazardous contaminant (HC) concentrations and loadings in dry weather sewage flows, and of runoff and CSO volumes and HC loadings. HAZPRED has been designed to estimate both average quantities (expected values) and the probability of specified events.

The paper presents an outline of the predictive techniques used in HAZPRED. A case study of a combined sewer catchment located in the Toronto area of Ontario is also presented. The case study illustrates model input requirements and the nature of model outputs.

INTRODUCTION

HAZPRED is an interactive microcomputer based model designed to predict the concentrations and loadings of selected hazardous contaminants (HCs) in dry weather sewage flow (DWF), urban stormwater runoff, and combined sewer overflow (CSO). The specific predictive capabilities included in HAZPRED are summarized in Table 1.

The selected hazardous contaminants included in HAZPRED are based upon the U.S. EPA list of 129 priority pollutants. The HCs which can be examined using HAZPRED include compounds falling within five classifications: Volatile Organics, Semi-Volatile Acid Extractables, Semi-Volatile Base/Neutral Extractables, Pesticides and PCBs, and Metals and Trace Elements.

TABLE 1. SUMMARY OF HAZPRED PREDICTIVE CAPABILITIES

Waste Stream	Predictions Performed
Dry Weather Sewage Flow	- HC concentrations in industrial wastewater - HC loadings in industrial wastewater - HC concentrations in dry weather sewage flow - HC loadings in dry weather sewage flow
Stormwater Runoff	- Average runoff event volume - Exceedance probability of runoff volumes - Average runoff HC concentrations - Average runoff HC loadings - Exceedance probability of runoff HC loadings - Average annual runoff volume - Average annual runoff HC loading
Combined Sewer Overflow	- Average CSO event volume - Exceedance probability of CSO volumes - Average CSO HC concentrations - Average CSO HC loadings - Exceedance probability of CSO HC loadings - Average annual CSO volume - Average annual CSO HC loading

MODEL STRUCTURE

Figure 1 presents the overall structure of HAPZRED. As is indicated, HAZPRED supports two levels of sophistication in the dry weather flow (DWF) model. The following discussion will focus on the Level II, or more sophisticated approach. Reference may be made to the HAZPRED documentation and to the User's Manual (Canviro, 1984, 1985, 1986) for additional details of Level I procedures.

CATCHMENT - SEWERAGE CONFIGURATION

The conceptual models of the catchment-drainage system used in HAPZRED for separate and combined sewerage are shown in Figures 2(a) and 2(b). A simple catchment model comprised of three parallel sub-catchments is employed. Open space areas are not treated per se but are assumed to be included in the pervious portion of each sub-catchment.

The parallel catchment model assumes:

i) that catchments are identically affected by regional climatology, and
ii) that each of the three sub-catchments responds independently to the same climatological input (no correlation of sub-catchment output probability distributions)

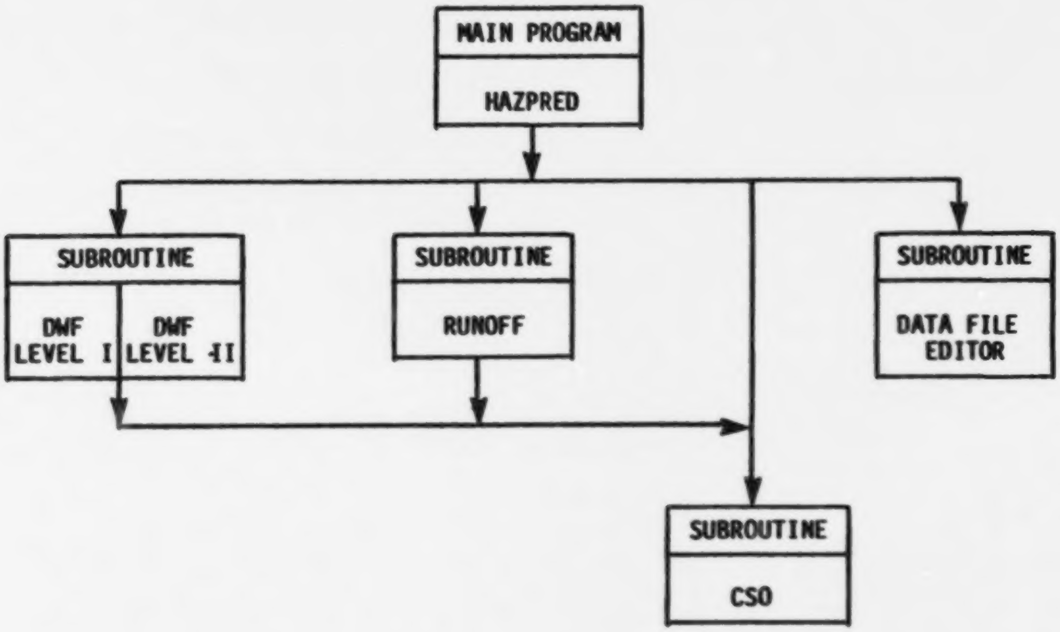


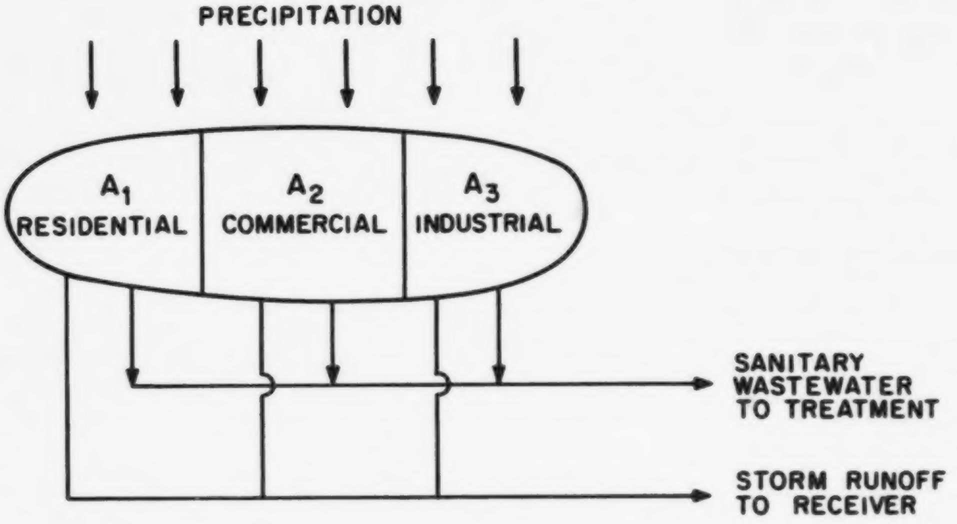
Figure 1. Schematic diagram showing general structure of HAZPRED.

Given these assumptions, the probability distributions of catchment outputs (flows and contaminant loads) are additive. Previous analysis of sewer flows by Adams and Gemmell (1973 a, b) have shown that the above may be justified since the impact of any covariance terms on probability estimates would be quite small and hence could be ignored.

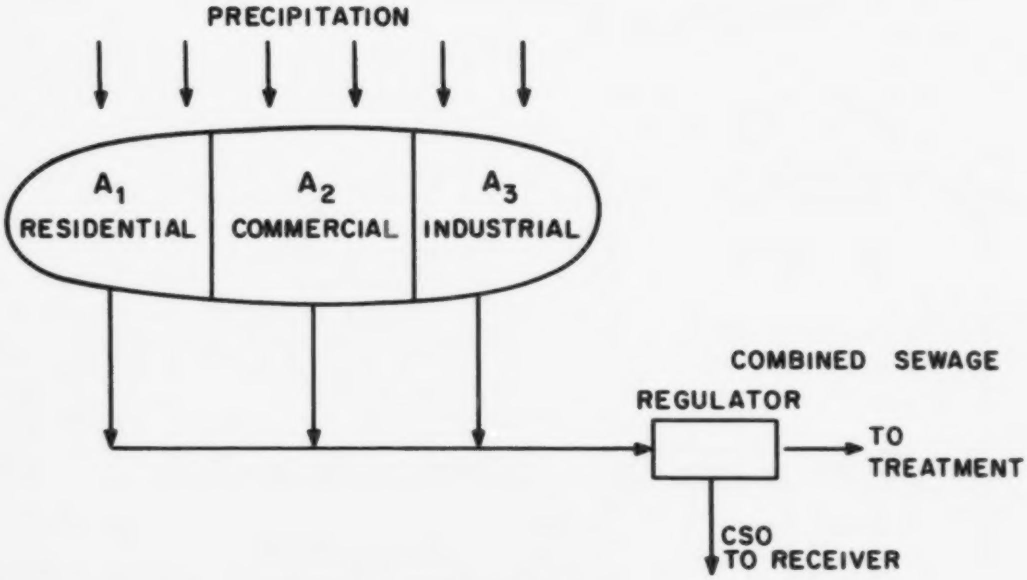
In dry weather (no precipitation), all land use areas contribute dry weather sewage flows which are transported to treatment by either sanitary or combined sewers. During periods of precipitation, rain falling on the catchment is transformed to runoff after accounting for losses due to depression storage and infiltration through pervious surfaces. Pollutants residing on catchment surfaces are washed off and transported with the runoff to the catchment outlet. In separate sewerage systems [Figure 2(a)], stormwater runoff is directly transmitted to the receiving water body. In combined sewerage systems [Figure 2(b], or nominally separate systems with a large wet weather component, stormwater runoff enters the sewer system and mixes with the sanitary wastewater. Combined sewage flows in excess of sewer or treatment plant capacity are overflowed at regulating structures; in the case of nominally separate systems, flows are by-passed at pumping stations or at the treatment plant.

ESTIMATION OF DRY WEATHER FLOW QUANTITY AND QUALITY

The quantity of dry weather sewage flow from a given area is dependent upon the nature of land use activity within the area and upon the influx of


PRECIPITATION
A_1
RESIDENTIAL
A_2
COMMERCIAL
A_3
INDUSTRIAL
SANITARY WASTEWATER TO TREATMENT
STORM RUNOFF TO RECEIVER

A. SEPARATE SEWERAGE


PRECIPITATION
A_1
RESIDENTIAL
A_2
COMMERCIAL
A_3
INDUSTRIAL
COMBINED SEWAGE
REGULATOR
TO TREATMENT
CSO
TO RECEIVER

B. COMBINED SEWERAGE

Figure 2. Schematic of HAZPRED catchment sewerage configuration.

groundwater through infiltration. For each land use, different techniques are required to develop the actual sewage flow components. A distinction should be made between flow estimates normally used for sanitary sewer or treatment facility design, and those that have application in a predictive model. Design values in general tend to be conservative, often producing higher than observed flow values and hence should not be employed.

Figure 3 shows the components making up sanitary sewage in dry weather and indicates the factors that affect the magnitude of each component.

Residential Wastewater Flow

The residential per capita wastewater flow rate is developed from metered water use data. Winter data is used since it reflects most accurately water returned to the sanitary sewer. Summer time activities such as lawn watering, car washing, etc., add to water use but not to sewer flows. Where household metering is not employed, water use data can be developed through the installation of test meters in selected households. Alternately, flow data from a suitable pumping facility can be used. Some allowance for water loss in distribution must be applied to pumping station data. An allowance of 10-15% is considered typical (AWWA, 1962) for "unaccounted" water (water lost in distribution).

Population data for a given catchment is developed from the most recent census figures. Since census and catchment boundaries may not match, the catchment population is estimated from unit area population of the census zone(s) and the catchment area.

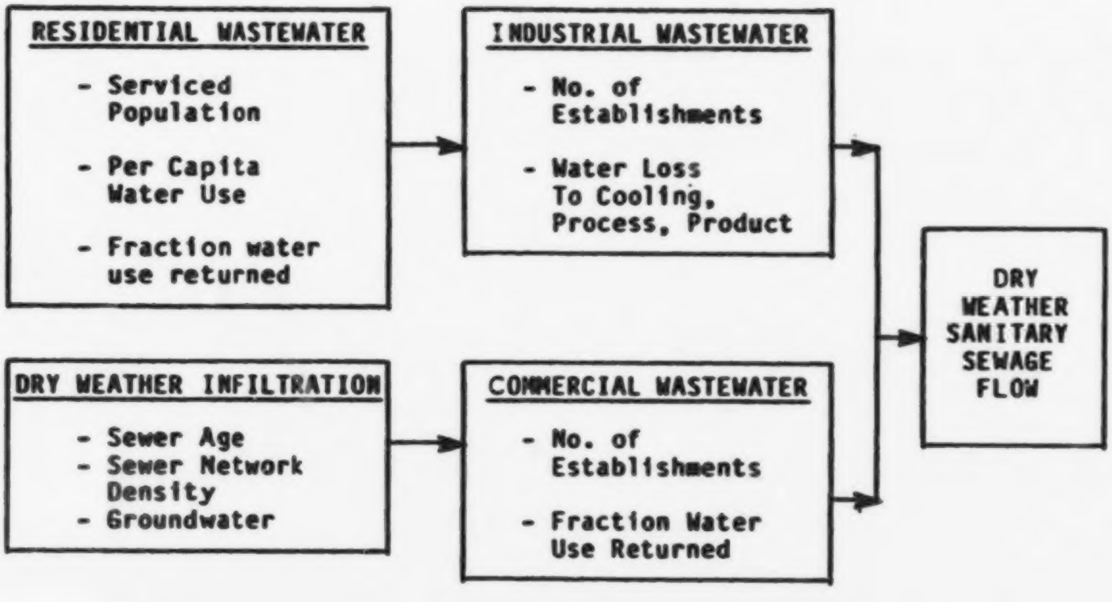

RESIDENTIAL WASTEWATER
- Serviced Population
- Per Capita Water Use
- Fraction water use returned
INDUSTRIAL WASTEWATER
- No. of Establishments
- Water Loss To Cooling, Process, Product
DRY WEATHER INFILTRATION
- Sewer Age
- Sewer Network Density
- Groundwater
COMMERCIAL WASTEWATER
- No. of Establishments
- Fraction Water Use Returned
DRY WEATHER SANITARY SEWAGE FLOW

Figure 3. Components of dry weather sanitary sewage

Commercial Wastewater Flow

In the HAZPRED model, it is assumed that the quantity of wastewater from commercial establishments is exactly equal to the commercial water usage. Water usage data are, again, ideally derived from individual metered records. Since the type of commercial establishments is not differentiated, commercial water usage is simply summed for a given catchment area.

Industrial Wastewater Flow

The relationship of water consumed to wastewater generated is considerably more complex for industrial establishments. Resident industries are first identified either through listing services (ie. Dunn and Bradstreet) or by field inspection. The latter method is more comprehensive, particularly for small establishments, but can be quite manpower intensive. Once the industrial inventory is complete, then water consumption records are accessed and a total industrial water usage is computed. The industries listed are now segregated into sublists based on industry type as defined by the Standard Industrial Category code (SIC code). In the event that industries are identified by field inspection, the SIC category is determined by direct contact with the industry. Major industrial water users (>1% of the total industrial water use for the catchment) are identified in each SIC sub-list and are then contacted in order to estimate the wastewater flow components of interest. In the case of combined sewer catchments, the process and sanitary flows as well as the cooling water flows are identified. For separated sewerage systems, only the process and sanitary component is normally needed. Although where municipalities allow cooling water discharge to sanitary sewers, estimation of both components will again be necessary. Occasionally, the wastewater or cooling water flow estimates will not be readily available from the industry. Best estimates of these flow components, based upon previous experience or knowledge of the industry, are then employed. Industrial wastewater flows generated by minor water users are estimated for each SIC category using total water usage data and a return factor of 0.85.

Dry Weather Infiltration

Dry weather infiltration is determined most accurately as the difference between metered dry weather flow data obtained at the catchment outlet, and the computed residential, commercial, and industrial wastewater flow components. Since monitored data is, however, only rarely available, DWI rates are usually estimated. Ontario experience (Cooper, 1984) has shown a reasonable infiltration allowance to be between 91 and 227 lcpd. The exact magnitude is determined subjectively from knowledge of the age of the sewer system and population.

Estimation of Dry Weather Sewage Hazardous Contaminant Concentrations

Estimates of HC concentrations in the residential and commercial components of wastewater are based on a 1979 U.S. EPA funded study carried out by A.D. Little (1979). The objective of the Little study was to determine the relative significance of source (residential, commercial or industrial) contributions of the EPA designated priority pollutants to publicly owned treatment works.

For industrial flows, HAZPRED, wherever possible, uses contaminant concentrations specific to the categories of industry present in the catchments. However, only a limited number of industrial effluents have to date received characterization. The most comprehensive industrial effluent data base is the EPA's Treatability Manual (U.S. EPA, 1980). The manual is the product of extensive monitoring and gives the incidence and effluent concentration of EPA priority pollutants for certain types of industry. Table 2 presents the industrial categories for which HC data is presently included in the HAZPRED model.

Contaminant concentrations in wastewater from all other industries are based upon average data obtained during the A.D. Little study.

WET WEATHER FLOW QUANTITY AND QUALITY

Analytical Models

Derived probability (analytical) models are used in HAZPRED to describe the wet weather behaviour of both separated and combined drainage systems. The models employ as input single parameter exponential distributions to describe the statistics of rainfall (ie. volume, intensity, duration, inter-event time) for a given locale. These distributions are mathematically transformed into the probability distributions of interest, such as runoff and CSO volumes, and pollutant loadings. The derived distributions are then employed to estimate distribution moments (ie. expected values) and the exceedance frequencies of selected events.

The theory and development of these models is well documented. Reference should be made to Adams and Bontje (1983) for details.

Runoff Quantity

A modified version of the coefficient method is used to describe the transformation of rainfall to runoff. The relationship between runoff and rainfall is expressed as:

$$Q_R = \phi (i - \phi) \tag{1}$$

where Q_R = runoff rate, mm-ha/hr
ϕ = modified runoff coefficient, ha
i = average rainfall intensity, mm/hr
ϕ = continuous abstraction rate, equivalent to depression storage, mm/hr

The value of the modified runoff coefficient, ϕ, is determined from the area weighted runoff coefficient for each sub-catchment. The overall expression for ϕ is given by:

TABLE 2. INDUSTRIAL CATEGORIES INCLUDED IN HAZPRED

SIC DESIGNATION	INDUSTRIAL DESCRIPTION
2200 - 2299	Textile Mill Products - yarn, thread, webbing manufacture - fabric manufacture and finishing
2821, 2823, 2824	Plastics Manufacture - plastic materials, synthetic resins - cellulosic man-made fibres and synthetic organic fibres
2834	Pharmaceuticals - other than perfumes and cosmetics
2841	Soaps and Detergents - manufacture soap, synthetic organic detergents, organic alkaline detergent, and crude and refined glycerine
2844	Cosmetic Preparations - preparations which function as skin treatments, excludes those used to enhance appearance
2851	Paint Manufacture - trade-sales paints, chemical coatings, varnishes, lacquers, epoxy coatings, and paint removers
2893	Printing Ink - letterpress, lithographic, flexographic and gravure inks, and varnish
3011 - 3079*	Rubber Processing - tire and tube manufacture and other rubber products
3111	Leather Tanning and Finishing - conversion of animal skins or hides to leather
3312	Coke By-Products - manufacture of light oils, tars, phenolates, etc. as by-products of coking
3321 - 3325	Foundries - melting, moulding or finishing of metals
3431	Procelain Enamelling - application of glass-like coating to steel, iron, aluminum or copper
3471	Electroplating - application of metallic surface coating by electrodeposition
3479	Coil Coating - painting of coiled sheet metals

* SIC-3079 included in this group since products are related

$$\phi \quad \sum_{j=1}^{3} \phi'_j \, A_j \tag{2}$$

where ϕ = modified runoff coefficient

ϕ'_j = runoff coefficient for the j^{th} sub-catchment

A_j = area of the j^{th} sub-catchment, ha

The values of ϕ_j for each sub-catchment are calculated from the runoff coefficients for pervious and impervious surfaces within the catchment, and the relative proportion of these surfaces within each sub-catchment.

The continuous abstraction rate, Φ , is related to the depression storage, average rainfall volume, and average rainfall intensity. The expression used to calculate Φ is :

$$\Phi = \frac{\zeta S_d}{\beta} \tag{3}$$

where Φ = continuous abstraction rate, mm/hr

ζ = reciprocal of average rainfall volume, 1/mm

S_d = available depression storage over the entire catchment, mm

β = reciprocal of average rainfall intensity, hr/mm

Runoff Quality

It is generally conceded that because of the complexity involved in modelling the transport of pollutants in stormwater runoff, a lumped parameter model is the most realistic representation which can be expected (Zison, 1980). Such a model is also consistent with the runoff model developed in the previous section.

A power law washoff function, relating pollutant washoff rate as observed at the catchment outlet, to the runoff rate, has been employed in the HAPRED model. The principal assumption of the washoff function is that no limitations exist on the quantity of pollutants which may reside on the catchment awaiting washoff. As a result, the rate of pollutant accumulation need not be considered.

The power law function describing washoff rate is given by:

$$W = Kq^n \tag{4}$$

where W = rate of pollutant washoff, gm/hr

K = washoff contant, gm/mm

q = runoff intensity, mm/hr

n = washoff exponent

The washoff constant, K, takes into account catchment hydrology and surface characteristics, and is calculated for each sub-catchment as follows:

$$K = \phi_j' A_j C_j / 100 \qquad (5)$$

where K = washoff constant, gm/mm

ϕ_j' = runoff coefficient for the j^{th} sub-catchment

A_j = area of the j^{th} sub-catchment, ha

C_j = average concentration of a single HC in runoff originating in the j^{th} sub-catchment, ug/L

Average hazardous contaminant concentrations in stormwater runoff (C_j) were obtained from an Ontario study conducted by Marsalek and Greck (1983) and from studies undertaken as part of the U.S. EPA Nationwide Urban Runoff Program (NURP) (U.S. EPA, 1982).

The study conducted by Marsalek and Greck (1983) examined the frequency, concentration and loadings of 51 selected persistent toxic substances in urban runoff in the Niagara River area. Samples of urban runoff were collected at sites in Fort Erie, Niagara Falls and Welland. Each site possessed a characteristic land use; of the eight permanent sampling stations, two were located in residential areas and six were located in industrial areas (sampling stations were not located in any commercial areas).

The preliminary NURP priority pollutant data base was used to obtain information on contaminant concentrations in runoff from commercial areas (U.S. EPA, 1982). A review of all NURP sampling sites identified five which listed 100% of land use to be commercial. Pollutant concentrations reported for each of these sites were abstracted from this data base.

CSO Quantity and Quality

Combined sewage quantity and quality are estimated in HAZPRED through simple mass and volume balance relationships based upon the relative proportions of runoff and dry weather sewage. Probability density functions derived from the mass and volume balances are then used to describe the distributions of CSO volumes and pollutant loads.

HAZPRED IMPLEMENTATION

The HAZPRED model is written in BASICA for the IBM PC microcomputer and compatibles, and requires the following hardware:

- IBM PC, PC/XT, PC/AT or compatible microcomputero MS-DOS 2.1 or higher
- 128 kilobytes of random access memory (128K of RAM)
- one double sided disk drive
- a keyboard
- monochrome or colour display monitor
- graphics adapter (to display piecharts)

In addition to the above requirements, HAZPRED will also support dot-matrix printers (EPSON FX-series) which can be used to generate hard copies of program text.

Three DOS system files are required to support the HAZPRED program; these are BASICA.COM, COMMAND.COM and GRAPHICS.COM. In order to run HAZPRED, the three system files must be resident in the same location as the HAZPRED program, either on the HAZPRED diskette or in the DOS sub-directory containing HAZPRED.

CASE STUDY

The HAZPRED model was employed to study a combined sewer catchment located in the City of York, Ontario (one of the lower tier municipalities making up Metropolitan Toronto). The catchment is 834 ha in area and is wholly serviced by combined sewers. The area is approximately 80% devoted to residential land-use with the remainder equally shared between industrial and commercial land-use.

DRY WEATHER PREDICTIONS

Dry Weather Wastewater Quantity

Dry weather flow quantities for the catchment required as HAZPRED input are presented in Table 3.

TABLE 3. DRY WEATHER FLOW WASTEWATER QUANTITIES

Total Wastewater Flow	(l/day) :	34,118,270
Residential Wastewater Flow	(l/day) :	15,116,960
Commercial Wastewater Flow	(l/day) :	48,890
Industrial Wastewater Flow	(l/day) :	503,220
Dry Weather Infiltration Flow	(l/day) :	18,449,200
Industrial Cooling Water Flow	(l/day) :	334,300

Details of the required industrial component flows are provided in Table 4. In this case, the catchment contains only seven significant industries of which two were represented by specific HAZPRED contaminant concentration data. (ie. 2851 - paint manufacture, 3011-3079 - rubber processing).

Dry Weather Wastewater Contaminant Concentrations and Loadings

Dry weather concentrations of priority pollutants predicted by HAZPRED are presented in Table 5. The < sign indicates that the predicted concentrations are below the Method Detection Limit, implying that the compounds would be detectable but not quantifiable with routine analytical techniques.

TABLE 4. YORK INDUSTRIAL FLOW COMPONENTS

Industrial Flow by SIC (1/day)

2200-2299	0	3011-3079	14,140
2821-2824	0	3111	0
2834	0	3312	0
2841	0	3321-3325	0
2844	0	3431	0
2851	10,680	3471	0
2893	0	3479	0
		Other SIC	144,100

Table 6 presents the predicted contaminant yearly dry weather loadings indicating both component loadings (ie. residential, commercial, industrial) and total loadings.

For the York catchment, depending upon the contaminant, either residential or industrial sources dominate the total loads. For example, in the case of the popular cleaning solvent tetrachloroethene, residential discharges are predicted as the major source of this substance. In contrast, acrylonitrile is predicted as exclusively originating from industrial sources.

WET WEATHER PREDICTIONS

Stormwater Runoff Volumes and Loadings

Input data needed for runoff and CSO predictions pertaining to both catchment and rainfall characteristics are presented in Table 7. A noteworthy feature of the catchment sewerage is the relatively high ratio of interception capacity to dry weather flow (6.2:1).

HAZPRED output for both storm and CSO predictions includes summary statistics as well as expected values and exceedance probabilities of loadings and volumes. Table 8 presents the summary runoff statistics for the catchment.

In total, 72 of the 89 rainfall events are of sufficient magnitude to produce runoff. The residential area, as may be expected, contributes most to runoff volumes. In addition to average runoff characteristics, HAZPRED provides the exceedance probabilities of annual runoff volumes, which are presented in Table 9.

These data may be generated for any range of desired volumes and may be used in a number of ways, including calculation of the return period of various volumetric events, according to the following equation:

$$T_R = \frac{1}{\theta\, p[V > V_o]} \tag{6}$$

TABLE 5. YORK OVERALL CONTAMINANT CONCENTRATION PREDICTIONS

Contaminant		Concentration (ug/l)
Carbon tetrachloride	<	1.00
Chloroform		1.44
Toluene		2.20
Acrylonitrile		2.49
Benzene	<	1.00
Chloroethane	<	5.00
Ethylbenzene	<	1.00
Methylene chloride	<	1.00
Tetrachloroethene		3.19
1,1,2-trichloroethane	<	1.00
1,2-dichloropropane	<	1.00
Trichloroethene	<	1.50
Phenol	<	10.00
Butyl benzyl phthalate	<	10.00
Diethyl-phthalate	<	10.00
Nitrobenzene	<	15.00
1,2-dichlorobenzene		1.25
Naphthalene	<	10.00
Bis(2-ethylhexyl)phthalate	<	10.00
Di-N-butyl phthalate	<	10.00
Antimony	<	2.00
Arsenic	<	3.00
Cadmium	<	2.00
Chromium	<	33.50
Copper		32.76
Lead		44.96
Mercury	<	1.50
Nickel	<	15.50
Selenium	<	3.00
Silver	<	2.00
Zinc		103.52

where T_R = average return period (years)
θ = average number of rainfall events/year
$p[V > V_o]$ = the probability of observing a runoff volume greater than V_o

For example, for the York catchment, a runoff volume of 10,000 m^3 has an average return period of 0.03 years, or approximately 1.5 weeks.

HAZPRED also predicts the expected values and exceedance probabilities of runoff pollutant loadings, both per average event and per average year.

Table 10 gives the expected values of annual runoff loadings for various contaminants, from the sub-areas of the catchment, and for the catchment as a whole. As was the case with the dry weather sewage, the different land-use areas contribute varying amounts of a given contaminant to the total annual mass.

CSO Volumes and Loads

HAZPRED summary statistics for combined sewer overflow events are presented in Table 11. As is evident, the large interception capacity results in a moderate number of annual overflow events.

TABLE 6. YORK PREDICTED DRY WEATHER FLOW YEARLY LOADINGS

Contaminant	Res (kg/yr)	Com (kg/yr)	Ind (kg/yr)	Total (kg/yr)
Carbon tetrachloride	0.00	0.00	1.72	1.72
Chloroform	16.55	0.10	0.84	17.49
Toluene	14.35	0.16	9.01	23.52
Acrylonitrile	0.00	0.00	21.63	21.63
Benzene	1.10	0.04	3.42	4.57
Chloroethane	0.00	0.00	8.91	8.91
Ethylbezene	2.21	0.04	6.97	9.28
Methylene chloride	0.00	0.00	2.22	2.22
Tetrachloroethene	34.76	0.31	3.19	38.27
1,1,2-trichloroethane	0.00	0.00	0.01	0.01
1,2-dichloropropane	0.00	0.00	0.03	0.03
trichloroethene	2.21	0.19	1.07	3.47
Phenol	32.00	0.07	5.96	38.03
Butyl benzyl phthalate	37.52	0.16	6.18	43.86
Diethyl-phthalate	54.07	0.08	0.00	54.16
Nitrobezene	0.00	0.00	0.30	0.30
1,2-dichlorobezene	15.45	0.11	0.00	15.56
Naphthalene	11.59	0.04	2.01	13.64
Bis(2-ethylhexyl)phthalate	37.52	0.11	2.99	40.62
Di-N-butyl phthalate	49.66	.17	3.17	53.00
Antimony	14.90	0.00	0.13	15.03
Arsenic	26.48	0.04	0.26	26.78
Cadmium	9.93	0.01	1.36	11.30
Chromium	89.94	0.83	27.76	118.53
Copper	397.83	0.80	6.43	405.06
Lead	536.87	0.73	15.48	553.08
Mercury	2.21	0.01	3.07	5.29
Nickel	23.17	0.18	5.10	28.46
Selenium	20.97	0.05	0.07	21.08
Silver	12.14	0.04	5.54	17.72
Zinc	1,180.79	2.03	73.98	1,256.79

TABLE 7. YORK CATCHMENT AND PRECIPITATION CHARACTERISTICS

Total Area (ha): 834

Combined Sewer Interception Capacity (l/d): 209,950,000

Coefficients

	Runoff	Depression Storage
Pervious:	0.25	2.0 mm
Impervious:	0.90	0.25 mm

Percentage Impervious Area

Residential (%): 45.5
Commercial (%): 71.6
Industrial (%): 84.8

Precipitation Characteristics

Reciprocal Average Intensity - (hr/mm): 0.716
Reciprocal Average Volume - (1/mm): 0.193
Reciprocal Average Duration - (1/hr): 0.288
Average No. of Annual Rainfall Events - : 89.4

Both expected values and exceedance probabilities of CSO contaminant loadings and volumes are available as HAZPRED output options. Table 12 illustrates the average values of annual CSO loads including the runoff and dry weather flow contributions.

HAZPRED has, as well, graphic output capability in the form of pie charts which present for specific contaminants the relative fractions originating from sewage and runoff. Figure 4 shows the pie chart for the York catchment representing total PCBs.

TABLE 8. PREDICTED YORK RUNOFF STATISTICS

Average Annual Number of Rainfall Events : 89
Average Annual Number of Runoff Events : 72

Average Annual Hours of Runoff : 251

Average Runoff Volume per Event
  Residential Runoff Volume (m^3) : 13,030
  Commercial Runoff Volume (m^3) : 2,105
  Industrial Runoff Volume (m^3) : 2,096

  Total Runoff Volume from Catchment (m^3): 17,233

Total Annual Runoff Volume (m^3) : 1,540,661

TABLE 9. PREDICTED YORK ANNUAL RUNOFF VOLUME EXCEEDANCE PROBABILITIES

Total Volume (m^3)	Probability of Exceedance
1	0.81
1,000	0.70
2,000	0.62
3,000	0.57
4,000	0.53
5,000	0.50
6,000	0.47
7,000	0.44
8,000	0.41
9,000	0.39
10,000	0.37

TABLE 10. PREDICTED YORK ANNUAL RUNOFF POLLUTANT LOADINGS

Contaminant	Res (gm/y)	Com (gm/y)	Ind (gm/y)	Total (gm/y)
1,2-Dichlorobenzene	30.06	0.00	7.91	37.97
1,2,4-Trichlorobenzene	2.56	0.00	0.52	3.09
1,3-Dichlorobenzene	3.61	0.00	0.97	4.59
1,4-Dichlorobenzene	5.94	0.00	0.86	6.80
Hexachlorobenzene	0.93	0.00	0.09	1.03
A-Endosulfan	1.51	4.95	0.13	6.60
B-Endosulfan	1.40	0.00	0.28	1.68
4,4-DDT	1.28	6.65	0.21	8.13
4,4-DDE	0.35	0.00	0.06	0.41
Aldrin	0.35	0.00	0.06	0.41
A-BHC	19.22	12.56	2.49	34.27
Chlordane	1.40	6.59	0.09	8.08
Dieldrin	0.82	6.46	0.11	7.39
Endrin	1.05	0.00	0.43	1.48
Heptachlor	0.47	0.00	0.07	0.54
Heptachlor epoxide	1.05	0.00	0.17	1.22
Total PCB	13.86	0.00	2.23	16.09
Arsenic	2,310.47	1,004.11	562.84	3,877.42
Cadmium	999.89	0.00	326.44	1,326.33
Chromium	1,106.72	0.00	433.01	1,539.72
Copper	10,096.39	8,378.14	2,308.96	20,783.49
Lead	5,000.03	37,390.96	2,144.96	44,535.95
Mercury	6,582.05	18.83	1,159.54	7,760.41
Nickel	6,989.78	596.26	1,634.76	9,220.80
Selenium	1,766.90	125.58	661.67	2,554.14
Zinc	31,745.26	135,179.80	13,442.51	180,367.60

TABLE 11. PREDICTED YORK CSO SUMMARY STATISTICS

Average Annual Number of Rainfall Events : 89
Average Annual Number of Runoff Events : 72
Average Annual Number of Overflow Events : 17

Average Annual Hours of Runoff : 251
Average Annual Hours of Overflow : 60
Average Overflow Volume per Event:
  Contribution from DWF (m^3) : 269
  Contribution from Runoff (m^3) : 4,833

Total Overflow Volume (m^3) : 5,102

Total Annual Overflow Volume (m^3)* : 456,186

TABLE 12. PREDICTED YORK ANNUAL CSO POLLUTANT LOADINGS

	DWF Contr. (gm/y)	Runoff Contr. (gm/y)	CSO Loading (gm/y)
1,2-dichlorobenzene	10.16	43.85	54.00
1,2,4-trichlorobenzene	0.16	0.69	0.84
1,3-dichlorobenzene	0.24	1.02	1.25
1,4-dichlorobenzene	0.35	1.51	1.86
Hexachlorobenzene	0.05	0.23	0.28
A-endosulfan	0.34	1.46	1.80
B-endosulfan	0.09	0.37	0.46
4,4-DDT	0.42	1.81	2.22
4,4-DDE	0.02	0.09	0.11
Aldrin	0.02	0.09	0.11
A-BHC	1.76	7.61	9.37
Chlordane	0.42	1.79	2.21
Dieldrin	0.38	1.64	2.02
Endrin	0.08	0.33	0.40
Heptachlor	0.03	0.12	0.15
Heptachlor epoxide	0.06	0.27	0.33
Total PCB	0.83	3.57	4.40
Arsenic	219.09	945.85	1,164.94
Cadmium	81.33	351.10	432.43
Chromium	298.81	1,289.99	1,588.80
Copper	1,283.75	5,542.02	6,825.76
Lead	2,585.38	11,161.24	13,746.62
Mercury	408.98	1,765.57	2,174.55
Nickel	575.87	2,486.06	3,061.93
Selenium	151.03	652.03	803.06
Zinc	9,955.49	42,978.51	52,934.00

HAZPRED STATUS

At present, the dry weather portion of the model has been verified in two Ontario catchments. It is planned to carry out further verification (dry weather only) in nine additional Ontario catchments. Wet weather data collection in three Toronto area catchments will be completed this year and will be used in connection with simulation modelling to verify the wet weather model alogrithms.

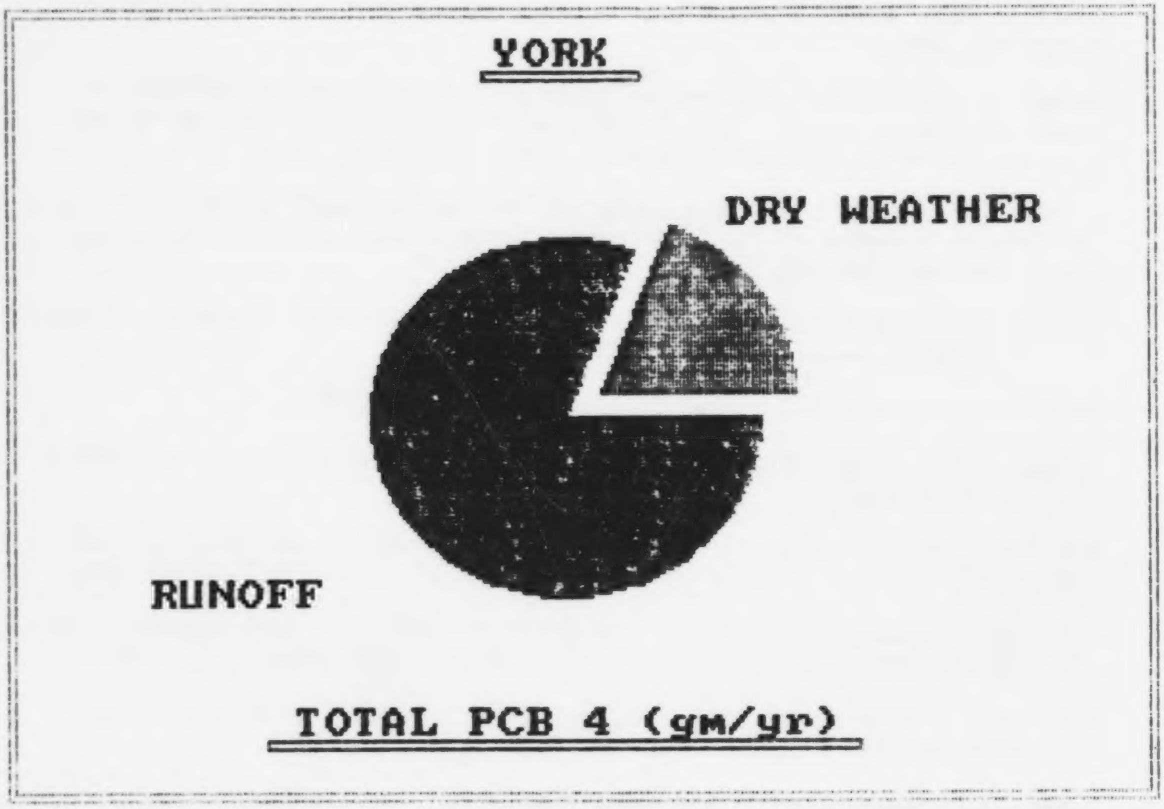


Figure 4. Total PCB loading distribution for the York catchment.

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ALTERNATIVE CALIBRATION OF THE QUAL-TX MODEL FOR THE UPPER TRINITY RIVER

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ABSTRACT

The objective of model calibration is to establish the values of free parameters by forcing an agreement between model predictions and field measurements. In water quality modeling, the kinetic rate coefficients are usually the model parameters of concern. It is frequently the case, however, that there are more model parameters requiring definition than there are data sets available. In this study, the QUAL-TX model, a version of QUAL-II, was calibrated for a 210 mile portion of the Upper Trinity River which flows through and south of the major metropolitan areas of Dallas and Fort Worth, Texas, and into which five major wastewater treatment plants discharge a combined average daily flow of over 400 MGD.

Only one set of field data was available that was satisfactory for model calibration, namely an intensive survey performed by the Texas Water Commission. To emphasize the role of subjective judgement in choosing coefficients when there is a lack of direct measurement defining real river processes, two distinctly different calibrations were performed based upon the observed dissolved oxygen measurements. The first calibration was based upon the postulate that nitrification is largely responsible for dissolved oxygen depletion in the river, and employed a carbonaceous decay coefficient of 0.1 per day, a coefficient of nitrification ranging from 0.3 to 0.5 per day, and sediment oxygen demand rates approximately 0.1 gram/m^2/day throughout the river length. The second assumed that carbonaceous decay and sediment oxygen demands are more important than nitrification. For this calibration a carbonaceous BOD decay coefficient of 0.3 per day, a coefficient of nitrification of 0.1 per day, and sediment oxygen demand rates in the order of 2 grams/m^2/day applied principally in the urban areas, were assumed with all other coefficients the same as in the first calibration. No significant difference in goodness of fit between predicted and observed DO values was obtained between the two calibrations. Both postulates are plausible; however, the implications on control strategies of employing one set of coefficients over the other are quite different.

The first set would imply the need for advanced treatment with strict ammonia removal, while the second would require strict processes for carbonaceous BOD and sediment control, and perhaps even nonpoint source controls.

INTRODUCTION

The Upper Trinity River flows through and south of the major metropolitan areas of Dallas and Fort Worth, Texas. There are four major forks of the Upper Trinity River - all of which are highly regulated by reservoirs utilized for water supply by municipalities in the Dallas/Fort Worth area. The Clear Fork of the Trinity is regulated by Benbrook Lake, the West Fork by Lake Worth and Eagle Mountain Lake, the Elm Fork by Lakes Grapevine and Lewisville, and the East Fork by Lakes Ray Hubbard and Lavon. Numerous tributaries, a few of which are also controlled by reservoirs, complete the drainage basin system for the Upper Trinity. Figure 1 shows the major features of the Upper Trinity River basin system.

A water quality modelling study was performed covering the 210 mile portion of the river from Beach Street in Fort Worth to Highway 31 at Trinidad, south of Dallas. Five major municipal treatment and approximately 30 minor municipal and industrial treatment plants discharge wastewater effluent into the modelled reach. In addition two major municipal plants discharge effluent into the East Fork which flows into the modelled reach. The major dischargers and their treatment capacities are shown in figure 2. Figure 2 also shows the locations of United States Geological Survey flow gaging stations and existing and planned water quality monitors.

BACKGROUND

Flows in the Trinity basin are highly variable. Under low flow conditions the river flow can consist completely of wastewater effluent. A frequency analysis performed by the Texas Department of Water Resources determined the 7 day 2 year low flow, less discharger return flows, to be 0.0 (zero) cfs at USGS gaging stations located in Grand Prairie between Fort Worth and Dallas, at Commerce Street in Dallas, and at Highway 34, 78 miles south of Commerce Street (1). At other times, immediately after rainstorms or later when flood water is released from upstream reservoirs, wastewater flows represent only a small percent of the total flow in the river.

Periods of low flow have always caused water supply and water quality problems for residents of the area. Early residents in this region were compelled to construct impoundments upstream of the cities to store water to improve water supply dependability. However, these impoundments have further aggravated the water quality problems by reducing the already insufficient natural flow in the river.

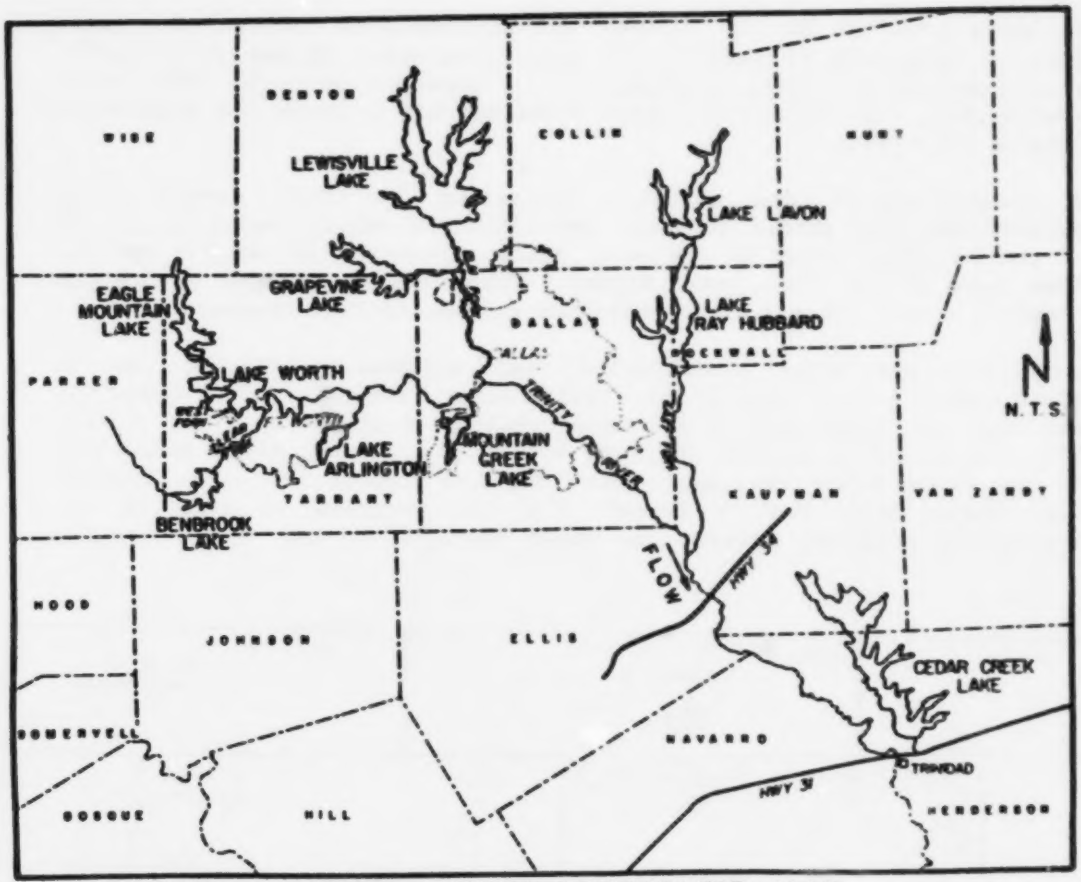
WISE
DENTON
COLLIN
HUNT
LEWISVILLE LAKE
LAKE LAVON
GRAPEVINE LAKE
EAGLE MOUNTAIN LAKE
DALLAS
LAKE RAY HUBBARD
ROCKWALL
PARKER
LAKE WORTH
LAKE ARLINGTON
MOUNTAIN CREEK LAKE
TRINITY RIVER
TARRANT
BENBROOK LAKE
KAUFMAN
VAN ZANDT
N
N.T.S.
FLOW
HWY 34
HOOD
JOHNSON
ELLIS
CEDAR CREEK LAKE
SOMERVELL
NAVARRO
TRINIDAD
HWY 31
HENDERSON
BOSQUE
HILL

FIG. I. TRINITY RIVER / RESERVOIR MAP

Low flow summer conditions, when most of the river flow is wastewater effluent, have caused oxygen depletion in the river near the cities for many years. The return flows have effectively created an artificial hydrological environment inviting fish to swim upstream in the effluent-augmented streamflows. Fish kills have occasionally been observed downstream after rise events associated with rainfall occurring in the Dallas/Fort Worth areas. Much speculation over the cause of the kills is ongoing even now and many studies are being planned to address this issue. At the present time it appears that the kills are rainfall related and occur within a certain range of rising stage in the river at Highway 34 south of Dallas. The automated water quality monitors show slugs of water having extremely low dissolved oxygen levels traveling downstream; however, the source of the low DO, either from nonpoint urban runoff, or resuspension of bottom sediments from wastewater effluent or nonpoint runoff deposits, is unknown. It is known that fish kills generally do not or have not been known to occur for rise events below 2300 cfs or above 10,000 cfs and generally fall in the 2300-3500 cfs range.

The water problems in the Trinity are difficult to solve. The small river is inadequate to support the large population of the river basin. Water supplies are being supplemented by transporting water from other river basins. The resulting return flows further increase the wastewater loads on the stream.

In recognition of the effluent domination and high summer water temperatures, the stream standard for dissolved oxygen has been set at 1.0 ppm for flows below 80 cfs above Beach street and set at 3.0 ppm for flows above 80 cfs above Beach Street. This standard changes to 5.0 ppm dissolved oxygen for all flow conditions at the Trinidad boundary.

Recognizing the major contribution that wastewater effluent has in maintaining the base flow in the Trinity River, the major plant operators have long been interested in the water quality of the river. In October, 1975, the cities of Dallas and Fort Worth, the Trinity River Authority, and the North Texas Municipal Water District formed the Upper Trinity River Basin Water Quality Compact for the purposes of carrying out cooperative programs related to water quality in the Upper Trinity

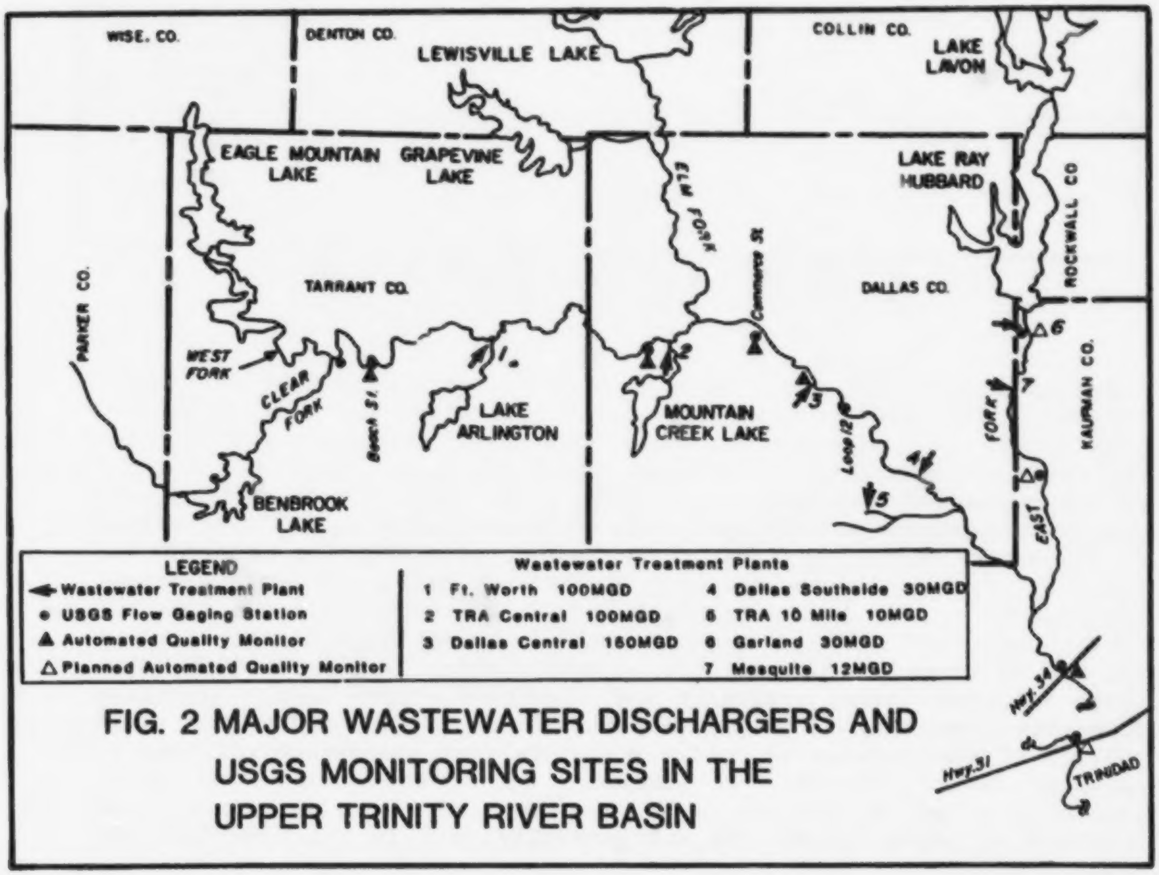
WISE. CO.
DENTON CO.
LEWISVILLE LAKE
COLLIN CO.
LAKE LAVON
EAGLE MOUNTAIN LAKE
GRAPEVINE LAKE
LAKE RAY HUBBARD
ROCKWALL CO
PARKER CO.
TARRANT CO.
ELM FORK
Commerce St.
DALLAS CO.
WEST FORK
CLEAR FORK
Beach St.
1
2
3
4
5
6
7
LAKE ARLINGTON
MOUNTAIN CREEK LAKE
Loop 12
FORK
EAST
KAUFMAN CO.
BENBROOK LAKE
Hwy. 34
Hwy. 31
TRINIDAD
LEGEND
Wastewater Treatment Plant
USGS Flow Gaging Station
Automated Quality Monitor
Planned Automated Quality Monitor
Wastewater Treatment Plants
1 Ft. Worth 100MGD
2 TRA Central 100MGD
3 Dallas Central 150MGD
4 Dallas Southside 30MGD
5 TRA 10 Mile 10MGD
6 Garland 30MGD
7 Mesquite 12MGD

FIG. 2 MAJOR WASTEWATER DISCHARGERS AND USGS MONITORING SITES IN THE UPPER TRINITY RIVER BASIN

River. Under this agreement four continuous automated monitors (CAMS) were installed to measure select water quality parameters. The Compact members and the US Geological Survey have had joint funding agreements for these monitors since 1977. A fifth monitor funded by the City of Dallas and the USGS was installed in February 1984. Arrangements have been made between the Compact and USGS for two additional monitors to be installed in spring of 1986, and the City of Garland and USGS should install an 8th monitor later in 1986.

These monitors continuously measure four water quality parameters, recording data at hourly intervals. These parameters are: dissolved oxygen, water temperature, specific conductance, and pH. Discharge measurements are also recorded. In addition to the continuous monitors, the Cities of Dallas and Fort Worth and the Trinity River Authority have long had various individual sampling activities within the modelled reach in which weekly or monthly grab samples were taken at different sites on the river. While the above data are quite useful in showing that an improvement in overall dissolved oxygen levels and other water quality parameters has taken place in the Trinity in recent years as the major wastewater plants have been upgraded, they are inadequate to develop an understanding of river processes.

In 1983 the Compact realized it was not enough just to know what the status of certain water quality parameters was in the Trinity. Rather, an understanding of river processes was desired. The Qual-Tx model, a version of Qual-II, was acquired from the Texas Water Commission (TWC, then the Texas Department of Water Resources) and implemented on the City of Dallas computer system. While the aforementioned sampling programs provided a wealth of water quality information, due to the uncoordinated grab-sample nature of the surveys and the fact that no BOD or NH_3-N data were acquired by the CAM monitors, inadequate data existed for calibrating the model. There were however, two intensive surveys completed by the TWC at the time of our study (2). The intensive surveys included sampling at 35 sites from Beach Street in Fort Worth to Highway 31 at Trinidad in addition to sampling the effluent at the major treatment plants. A composite sample was made from the four to five grab samples taken at each site spaced over a sixteen hour period from predawn to after dark. In-situ readings at the time of each grab sample were also recorded. The TWC generously made these data sets available to the Compact for use in studying the river. Under the guidance of Dr. George Ward of Espey Huston & Associates, the Compact attempted to utilize the data sets in calibrating the Qual-Tx model.

DATA SET REVIEW

The first step in employing these data sets was a review of the conditions under which they were obtained. The Qual-Tx model is designed to be applicable only to steady-state conditions. This means a condition whereby all components of the system, including streamflow, wasteloads, water temperature, etc., are steady in time throughout the modelled reach, and the dissolved oxygen profile has equilibrated to these steady components. For the data sets to be applicable for calibration and verification purposes they must represent a reasonable steady state condition.

The first and easiest check for steady state conditions for the two available data sets was a review of antecedent streamflows in the river. If flow was too highly variable immediately prior to or during the intensive survey sampling activities, then the sampling would not have been made under steady state conditions and the data sets would not be applicable for calibration. In fact this was found to be the case for one of the data sets.

Figure 3 shows the daily streamflow values at USGS gaging stations for the 30 days prior to and during the sampling for the September, 1982 survey. As it turns out the survey was conducted just a few days after flood releases from upstream reservoirs were discontinued. The declining flows shown on figure 3 are characteristic of gradual decreases in flood releases as reservoir levels approach the top of conservation storage. Further review showed that in fact very high releases had been continuously made for a full thirteen months immediately prior to the survey.

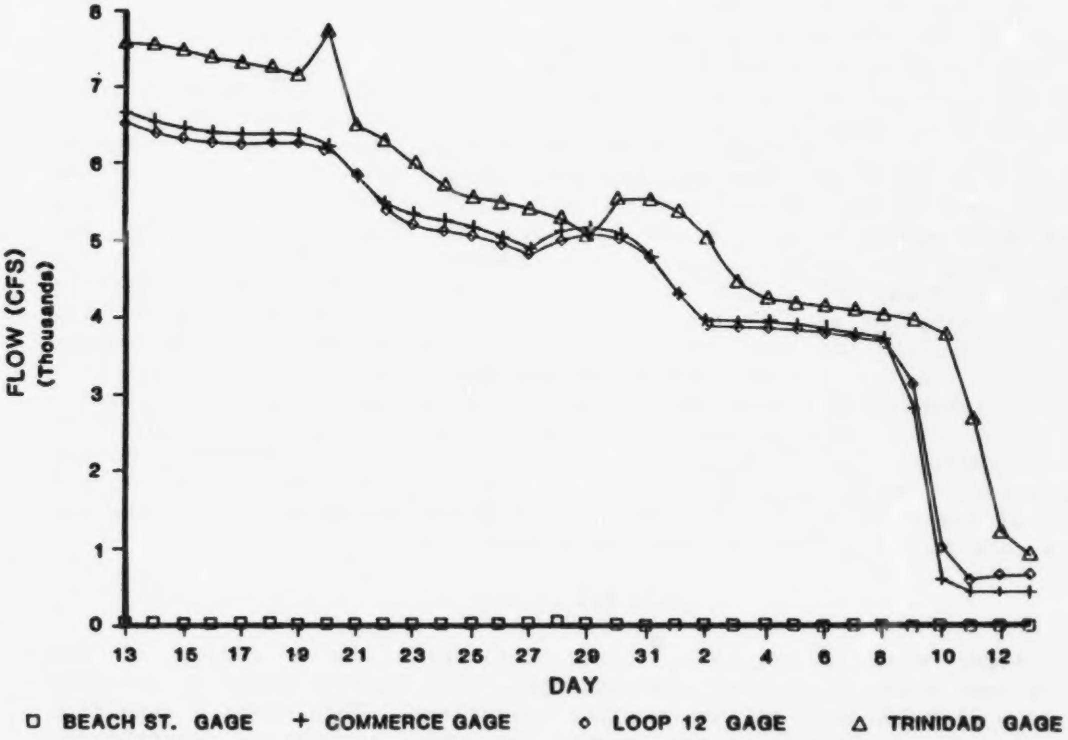

FIG. 3 USGS STREAMFLOW
13 AUG TO 13 SEPT 1982

While streamflow had steadied for two or three days before the survey, this was insufficient to ensure steady state hydrological conditions. For this to have occurred, steady flows should have been achieved throughout the model reach for a period at least equal to the travel time from the head to the end of the reach. This would have been on the order of 10 days for the normal expected low flows. The September, 1982 data set was therefore judged to be inapplicable for calibration purposes. Analysis then turned to the October, 1982 data set. Figure 4 shows streamflows for the 30 days during and prior to this survey. While some minor variations in flow were noticeable, the data set was judged applicable for calibration purposes.

CALIBRATIONS

Since there were more model parameters requiring definition than data sets available to ensure the assumed rate coefficients mirrored real world conditions, and to emphasize the importance of correct subjective judgement in choosing from a range of "reasonable" values, two sets of rate coefficients, both within ranges cited in the general literature

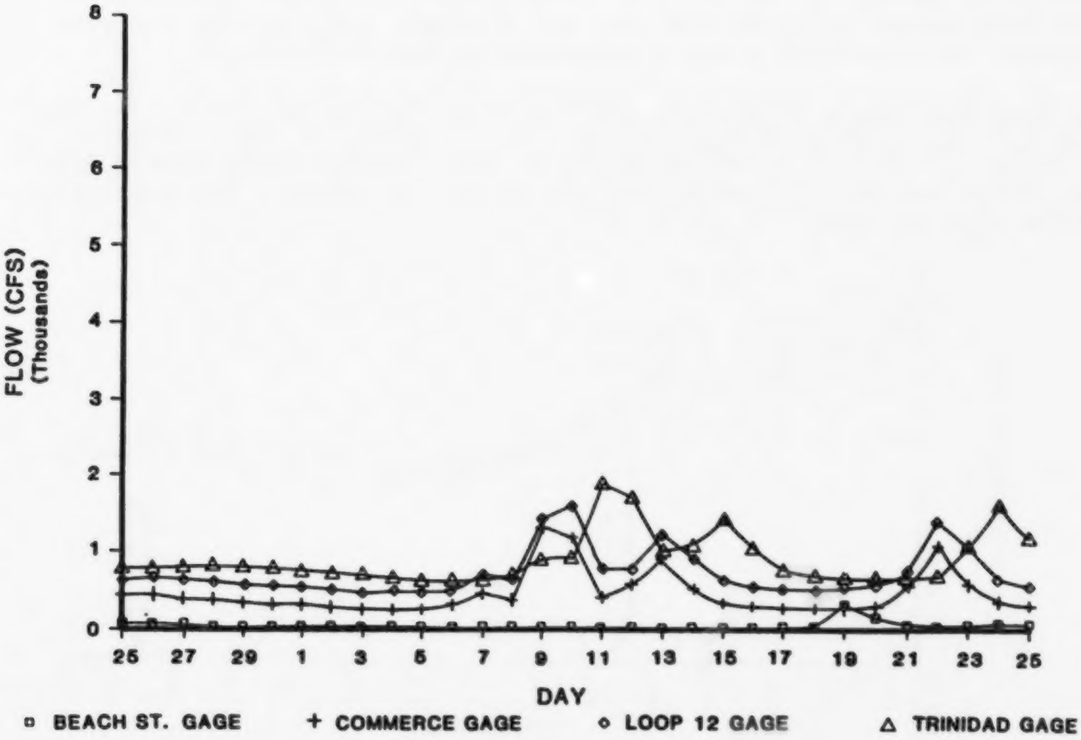

FIG.4 USGS STREAMFLOW
25 SEPT TO 25 OCT 1982

were utilized in developing two distinctly different calibrations based upon the observed dissolved oxygen measurements.

The first calibration assumed, at 20°C, a carbonaceous BOD decay rate of 0.1 per day, a coefficient for nitrification ranging from 0.3 to 0.5 per day, and a benthal oxygen demand on the order of 0.1 gram/m^2/day. This last coefficient would assume that benthal oxygen demand in the Trinity primarily originates with point source dischargers and the advanced waste treatment permits in effect for area operators (10 mg/L BOD, 15 mg/L TSS) keep these demands very low.

The second calibration assumed a carbonaceous BOD decay rate of 0.3 per day, a coefficient for nitrification of 0.1 per day and a benthal oxygen demand rate within the urban corridors on the order of 2 grams/m^2/day. The 2g/m^2/d is a moderate demand and its use would assume that there were sources of oxygen demanding deposits other than point sources. All other coefficients were unchanged between the two calibrations.

Figures 5, 6 and 7 display the measured dissolved oxygen values from the October, 1982 intensive survey. The vertical bars represent the range of values obtained throughout the survey and the small circles indicate the arithmetic average of the readings at each sample station. The modelled dissolved oxygen profiles from the two alternate calibrations are also plotted. Both profiles could be considered as good calibrations.

The Root Mean Square value, the square root of the average of the sums of the differences between the observed and the predicted values squared, provides one measure of the "goodness of fit". These values were 0.538 and 0.356 respectively indicating the second calibration was somewhat better than the first.

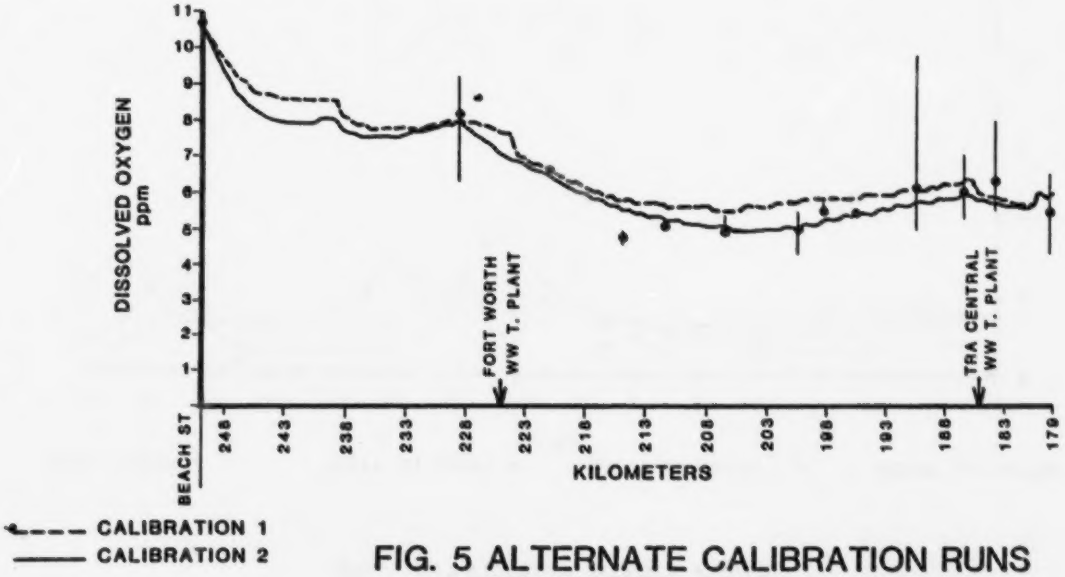

FIG. 5 ALTERNATE CALIBRATION RUNS COMPARED TO DATA POINTS

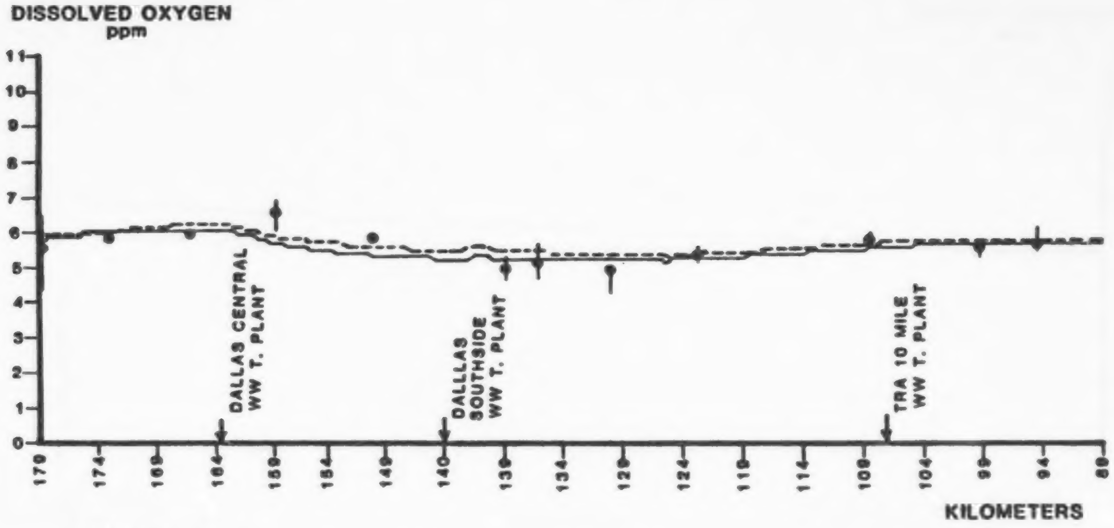

FIG. 6 ALTERNATE CALIBRATION RUNS COMPARED TO DATA POINTS

CONCLUSIONS

With two essentially equally good calibrations, which is better? The answer is that we cannot tell from the available information. Of course it was known at the outset that this was the case. The purpose of the exercise then was not to have a completely calibrated model which one could confidently employ in answering management decision questions. On the contrary, the purpose was to show that Trinity River processes are far from understood and that decisions made from one set of calibration assumptions could be very different from decisions made from the other set of calibration assumptions.

For example, a control strategy decision made from the first calibration, would call for advanced treatment with strict ammonia removal. In contrast, a strategy based on the second calibration, which emphasizes carbonaceous kinetics and sediment demands, would call for strict carbonaceous BOD and sediment controls, and perhaps even nonpoint source controls. Either strategy requires substantial capital investments. At this time the major plant operators on the Upper Trinity are committed to achieving 10 mg/L CBOD, 15 mg/L TSS, and 3 mg/L NH_3-N warm weather, 5 mg/L NH_3-N cool weather limitations as called for in the most recent wasteload allocation (3).

A concerted data collection effort is now underway to collect additional data sets to help establish "correct" coefficients for future modelling efforts. Future calibrations will center not only on predicted versus actual measurements of dissolved oxygen, but also on comparisons of BOD,

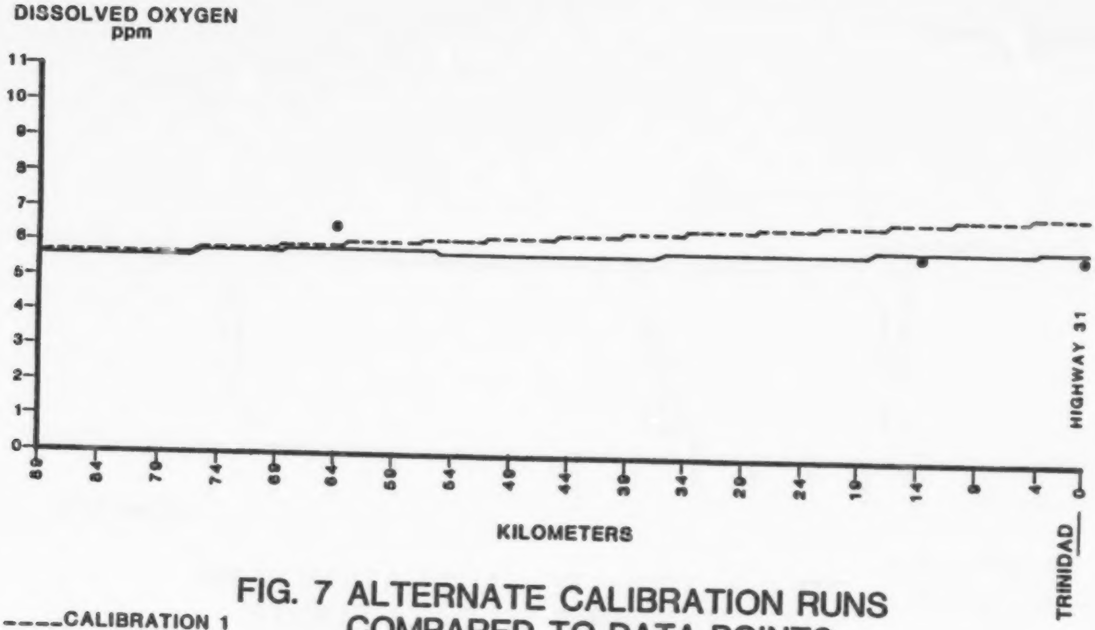

FIG. 7 ALTERNATE CALIBRATION RUNS COMPARED TO DATA POINTS

NH_3-N, NO_3-N+NO_2-N, and perhaps chlorophyll-a. Three intensive monitoring surveys have already been performed by Compact members under fairly steady state flow conditions. More will be completed as conditions permit. Additional special studies directed toward better definition of the nitrification and CBOD decay processes, an investigation of observed in-stream nitrogen losses, algal kinetics and photosynthesis/respiration, as well as a reanalysis of hydrology and hydraulics relationships are also planned.

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1. Texas Department of Water Resources. Waste Load Evaluation For The Upper Trinity River System In The Trinity River Basin. Texas Department of Water Resources, June 25, 1985. P.22.

2. Davis, J. W. Intensive Survey of the Trinity River Segment 0805. IS-53, Texas Department of Water Resources, June, 1983. 62pp.

3. Texas Department of Water Resources. Waste Load Evaluation For The Upper Trinity River System In The Trinity River Basin. Texas Department of Water Resources, June 25, 1985. 299pp.

The work described in this paper was not funded by the U.S. Environmental Protection Agency and therefore the contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

LOGNORMALITY OF POINT AND NON-POINT SOURCE POLLUTANT CONCENTRATIONS

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ABSTRACT

This paper presents a series of probability plots of water quality data from a variety of discharge sources. It is intended to provide a visual display of the appropriateness of characterizing the variable pollutant concentrations by a log normal distribution.

Representative examples of observed data that have been analyzed and plotted to test whether they can be treated as lognormally distributed random variables, are presented for data sets from the following applications:

Highway stormwater runoff

Combined sewer overflows

Urban runoff

Point Source discharges from POTW's

Agricultural runoff

Such examination suggests that a lognormal distribution either actually defines the underlying population of pollutant concentrations, or is at the least a satisfactory approximation for most environmental analyses.

INTRODUCTION

It has always been recognized that natural processes are variable. The value of ***quantifying*** this variability in some appropriate way has increased substantially in recent years. This is because many of the issues, problems, situations that the environmental engineering community is now dealing with are more effectively addressed in a probabalistic context.

A number of articles have appeared in the technical literature over the past five years or so, in which the authors report that the particular data they are dealing with has a log normal distribution. Almost invariably, such statements carry a qualifier. Some examples

> The log normal distribution was found to fit . . . . most consistently.
>
> (Three distributions). . . were found to be adequate, with log normal preferred because of ease of application.
>
> Visual examination . . . indicated that data were best described by a log normal distribution.
>
> Generally log normal distributions were observed for all but the data extremes.
>
> Overall, the . . . data tend to fit a log normal distribution.

Space constraints will nearly always preclude the inclusion of any significant number of distribution plots in a paper or report, so the reader has no independent basis for deciding how well, or how poorly, the general conclusion on lognormality really applies.

The simple objective of this paper is to present a set of probability plots for water quality data from a number of different applications, to provide the reader with a visual picture of the extent to which the sampled observations fit a log normal distribution.

During the inspection of the probability plots, it will be useful to bear in mind the following considerations. I submit that the important issue is not whether a specific data set can be reliably concluded to have a log normal distribution. The real issue is whether it is appropriate or reasonable to infer that the underlying population of events represented by the sample of observations is lognormally distributed.

Each data set is a small sample of the much larger population of values represented by the sample. The particular sample taken will be representative of the underlying population to an unknown extent, and may be fairly good or rather poor. There is no way to resolve this satisfactorily when it is the only sample available for inspection. However, when similar pollutant data are available for inspection from a large number of comparable sites, or for a variety of other pollutants at the same site, the information to guide the desired inferences is extended.

HIGHWAY STORMWATER RUNOFF

The probability distribution of event mean concentrations (EMC's) of four pollutants are shown for each of four highway sites. Three of the sites, Nashville, Milwaukee and Denver are urban highways. The Harrisburg site is in a rural setting. Figure 1 is for TSS, Figure 2 for Total N, Figure 3 is for Lead and Figure 4 is for Zinc. These data are from a study currently in progress for the Federal Highway Administration.

COMBINED SEWER OVERFLOWS

The probability distribution of event mean concentrations (EMC's) of BOD (Fig 5) and suspended solids (Fig 6) are shown for four CSO sites. Three of the sites, are in Richmond VA, the fourth in Toronto.

Figure 7 shows the distribution of the site median concentrations of BOD and TSS at the six CSO sites that were monitored in Richmond, and at 13 sites from 6 cities. All data is from the University of Florida Data Base.

URBAN RUNOFF

The probability distribution of event mean concentrations (EMC's) of Total P (Fig 8) and COD (Fig 9) are shown for six urban runoff sites. Figure 10 shows the distribution of the site median concentrations of Total P at 69 of the urban runoff sites that were monitored under the Nationwide Urban Runoff Program (NURP)

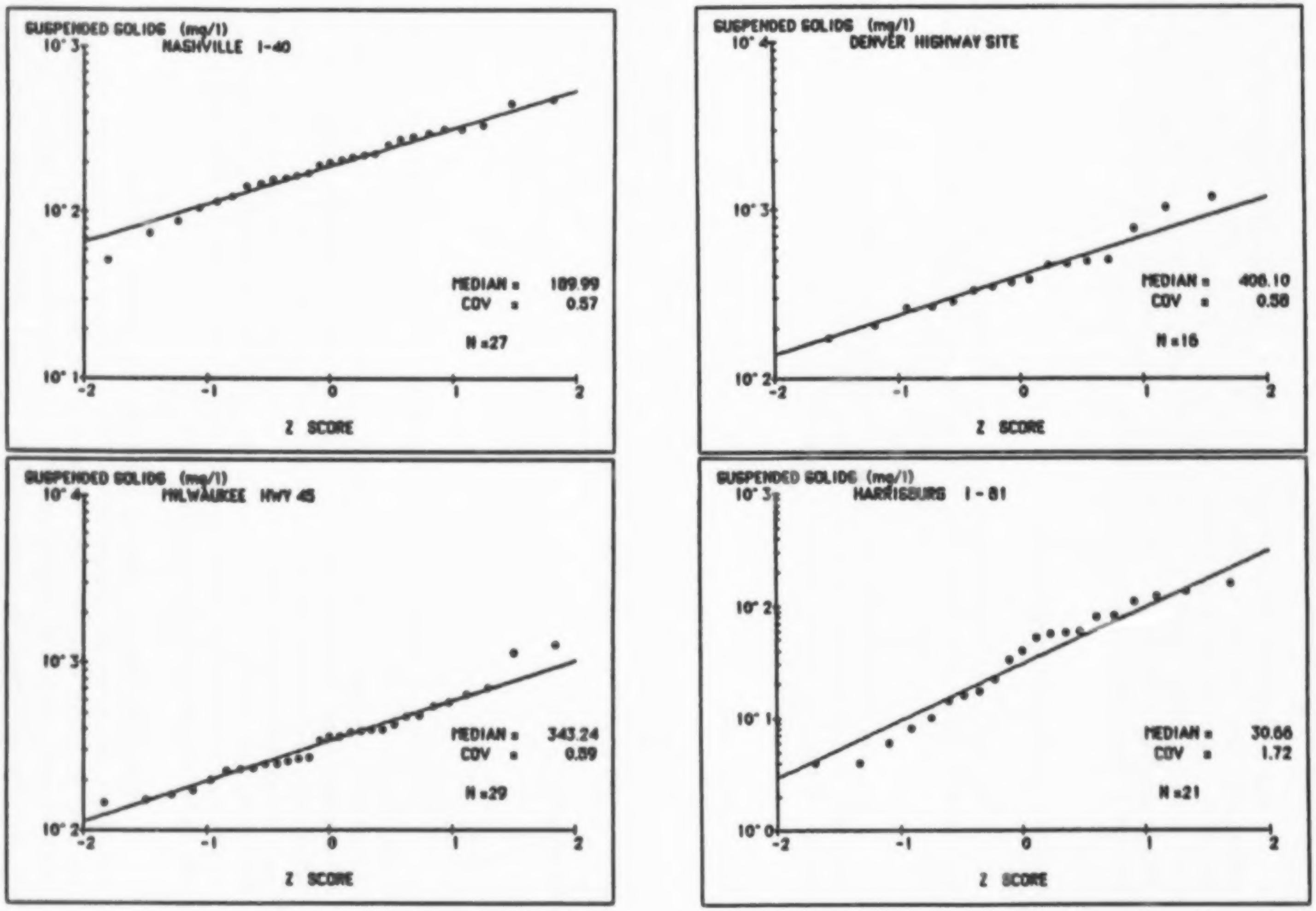
POINT SOURCE DISCHARGES - POTW's

There have been a number of studies over the past 6 or 8 years that have looked at the variability of daily average effluent concentrations from municipal sewage treatment plants. The authors have, as cited earlier, concluded that distributions of daily average concentrations are at least adequately approximated by a log normal distribution. I haven't had access to such daily values and have no plots to present to illustrate this level of detail. However the papers and reports do present summaries of the mean and coefficient of variation (COV) of daily values for a sample that generally covers one to three years of plant records.

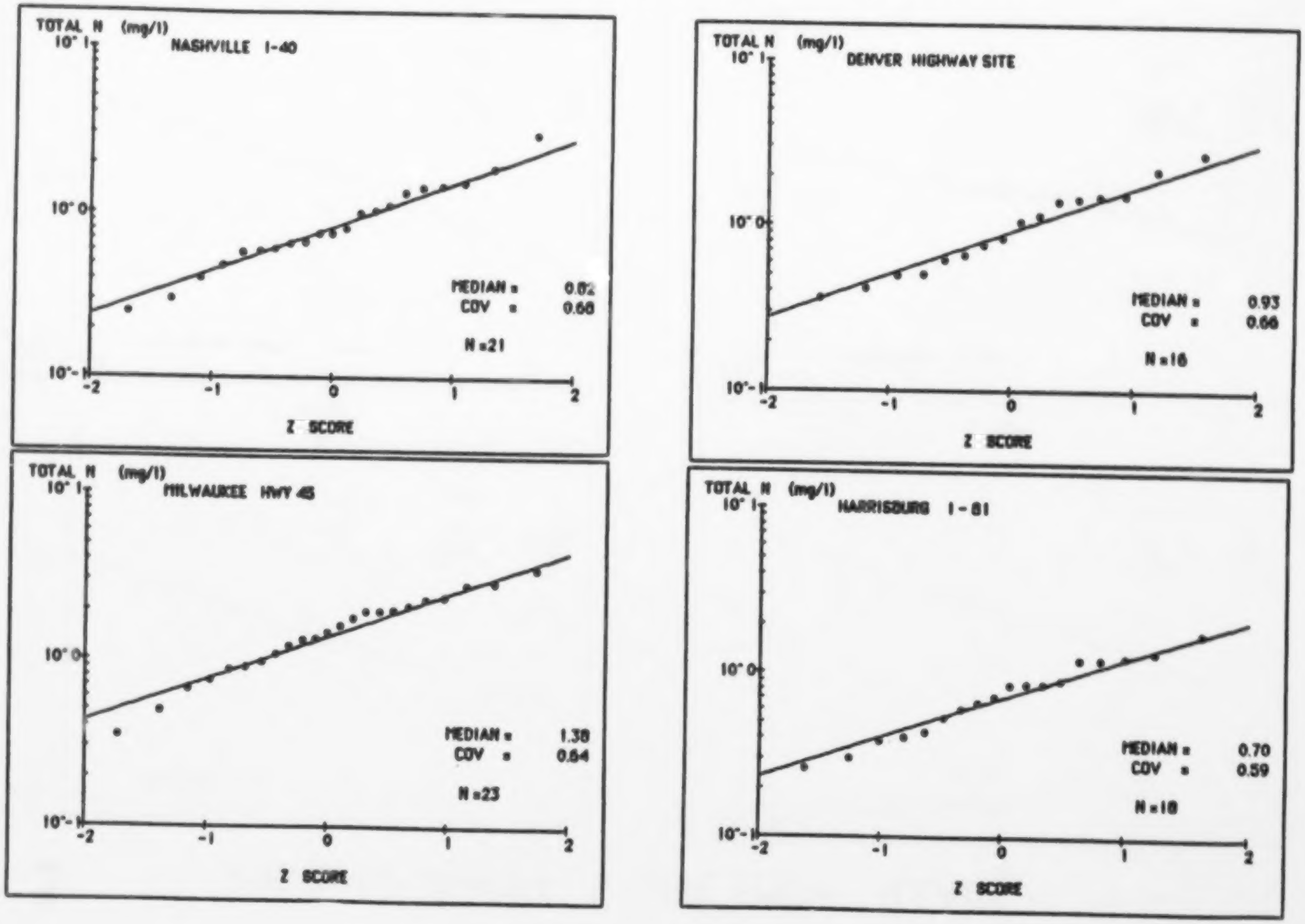
Figure 11 shows the distribution on mean influent concentrations of Cadmium for a large sample of POTW's. Figure 12 shows the distribution of the overall mean effluent BOD concentration, and the COV of daily values for 66 Conventional Activated Sludge plants. The correlation plot shows that there is no relation between the mean effluent that a particular plant produces and the degree of variability of daily concentration values - for all plants in this process category.

1

HIGHWAY RUNOFF - SITE EMC's (TSS)

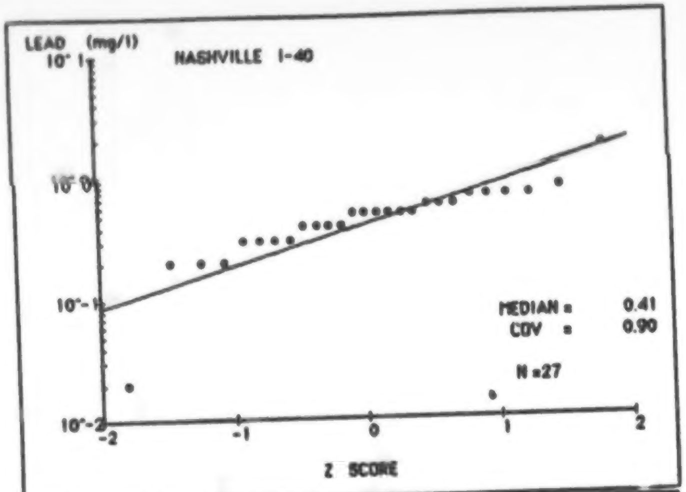


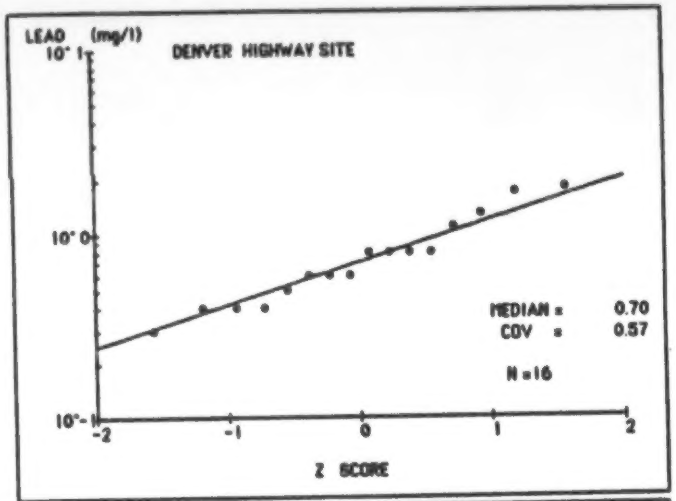
2 HIGHWAY RUNOFF - SITE EMC's (TOTAL N)

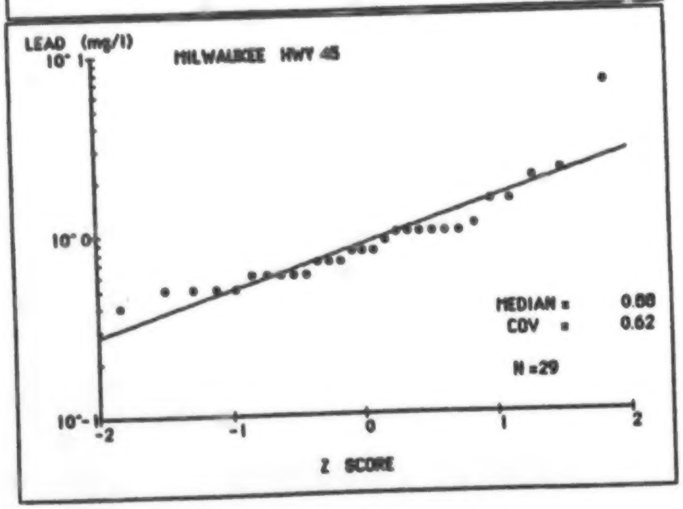


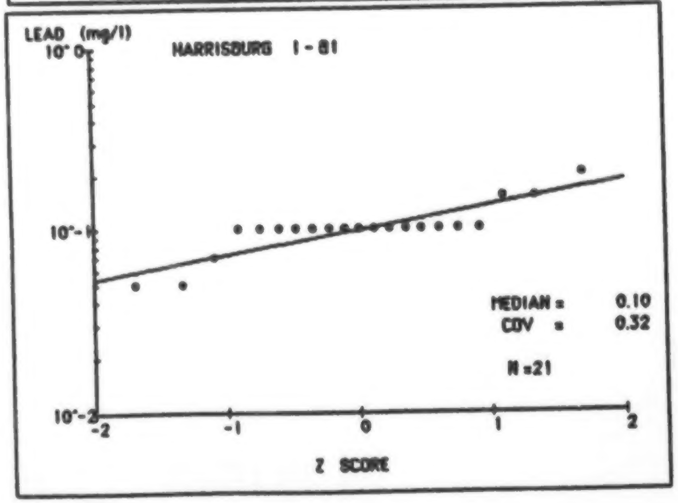
3

HIGHWAY RUNOFF - SITE EMC's (LEAD)

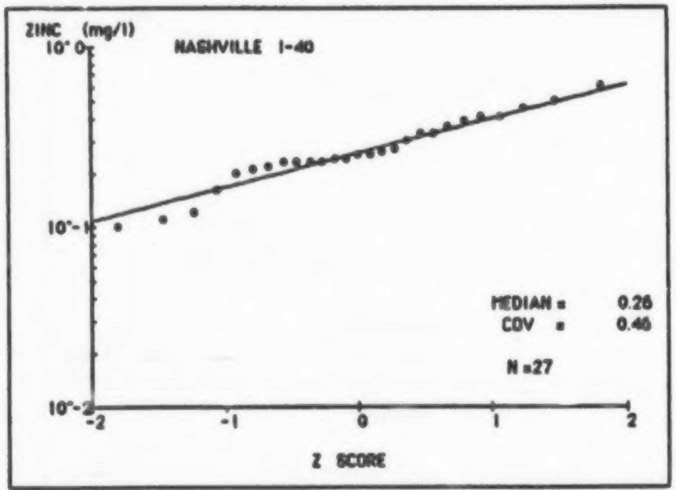

NASHVILLE I-40

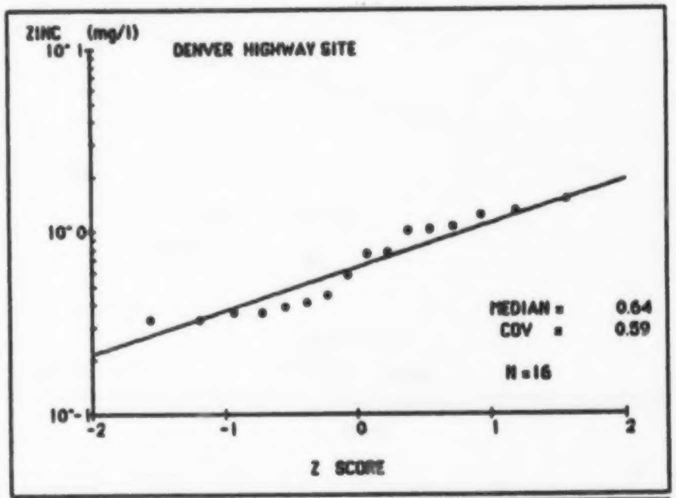

DENVER HIGHWAY SITE

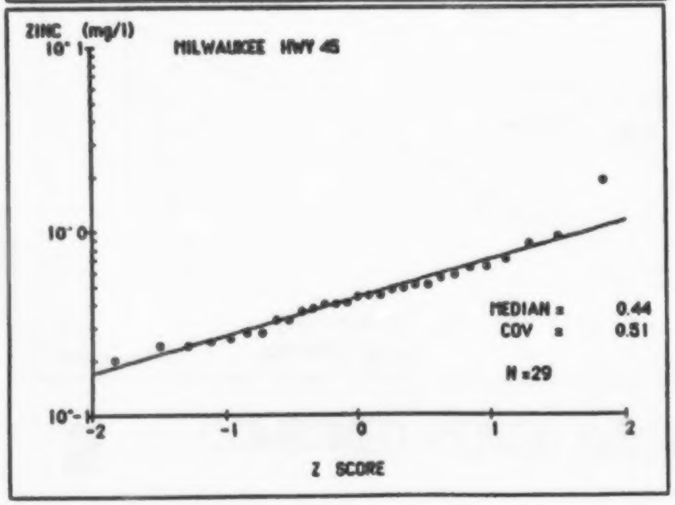

MILWAUKEE HWY 45

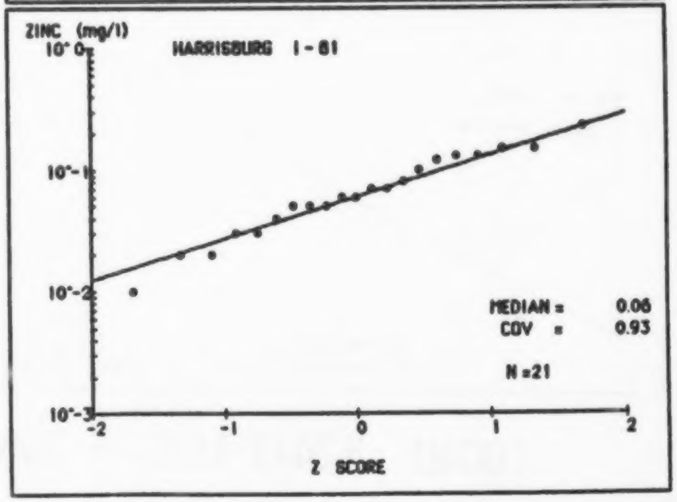

HARRISBURG I-81

4 HIGHWAY RUNOFF - SITE EMC's (ZINC)

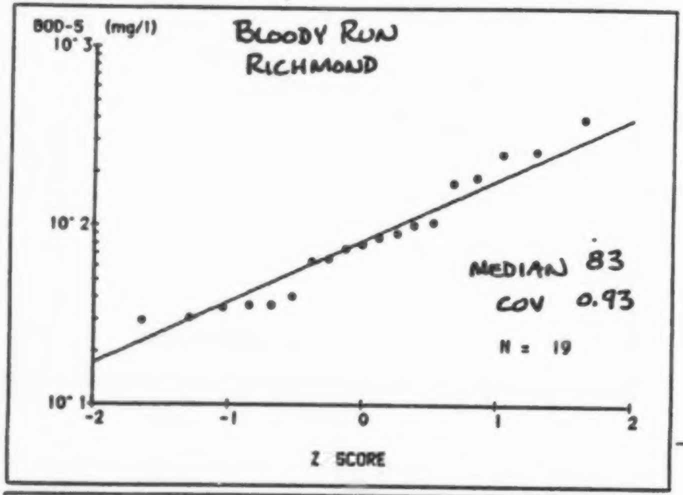


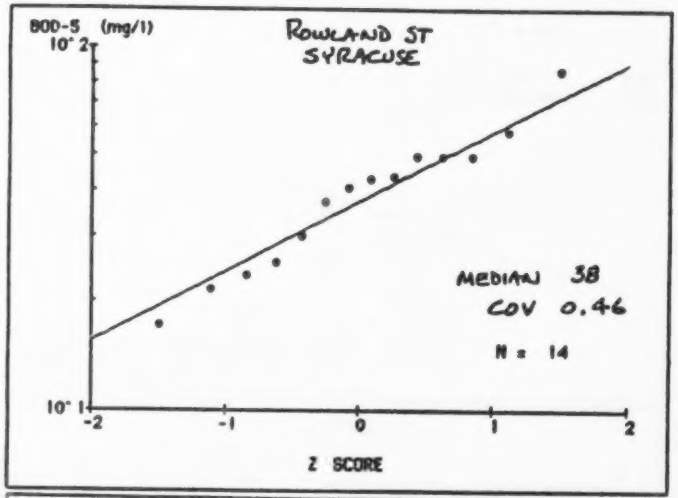


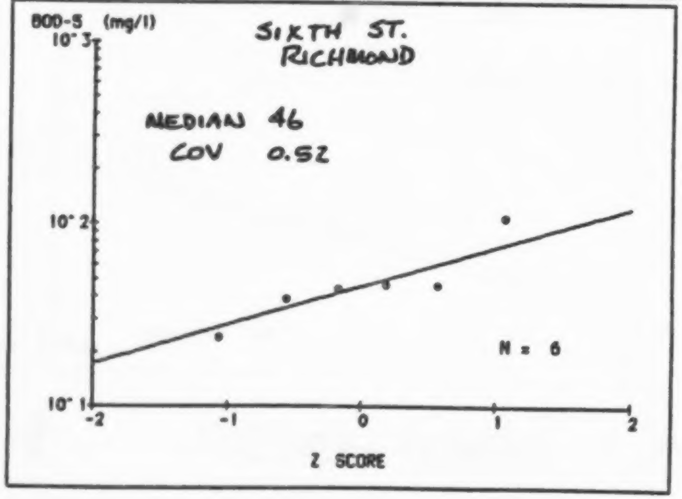


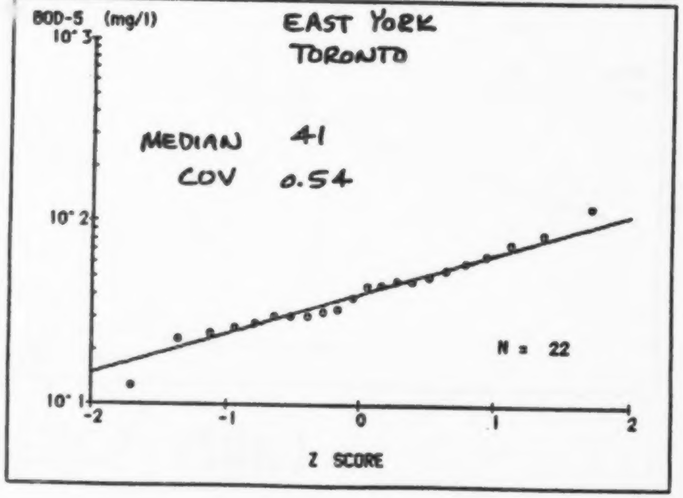


5 COMBINED SEWER OVERFLOWS - SITE EMC's (BOD)

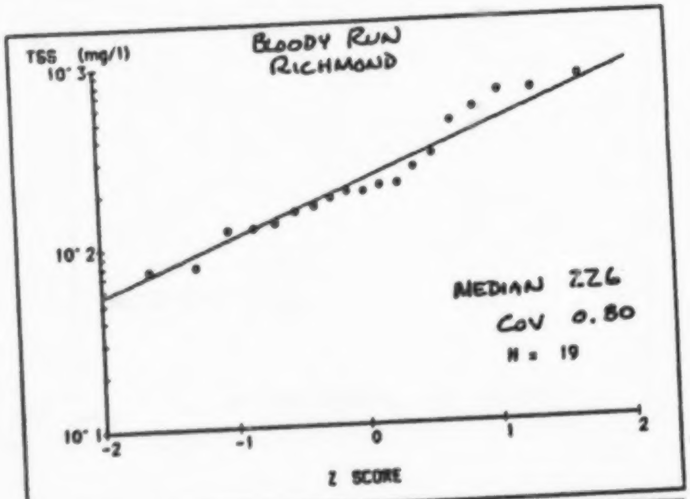


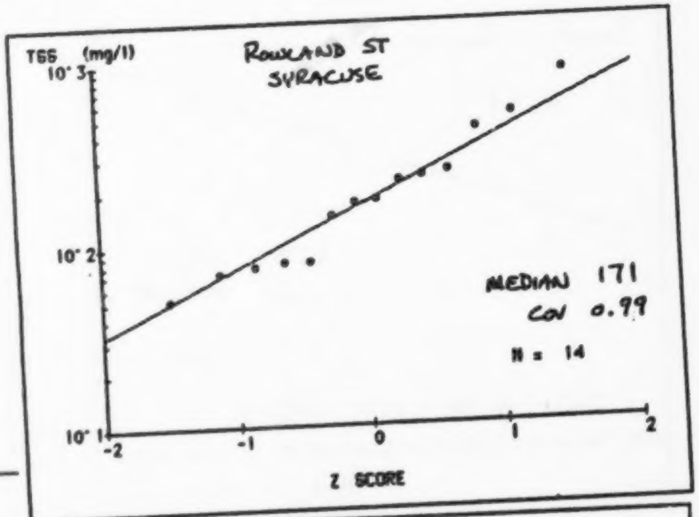


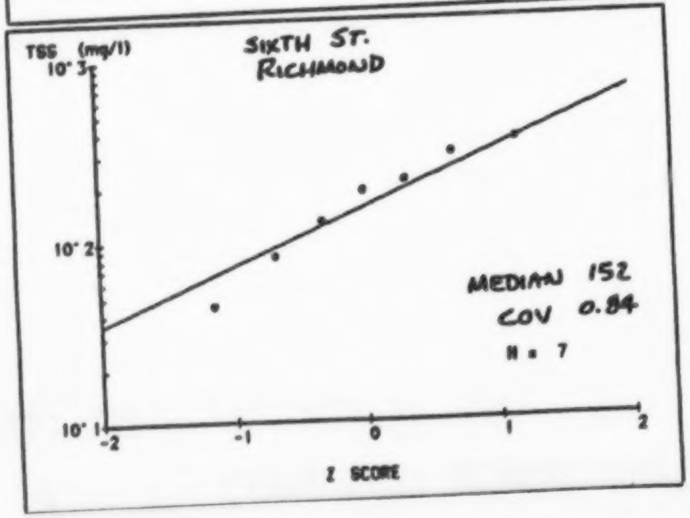


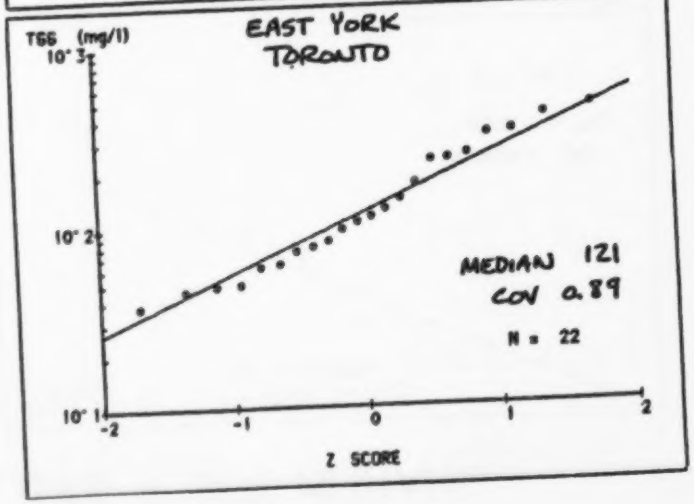


6 COMBINED SEWER OVERFLOWS - SITE EMC's (TSS)



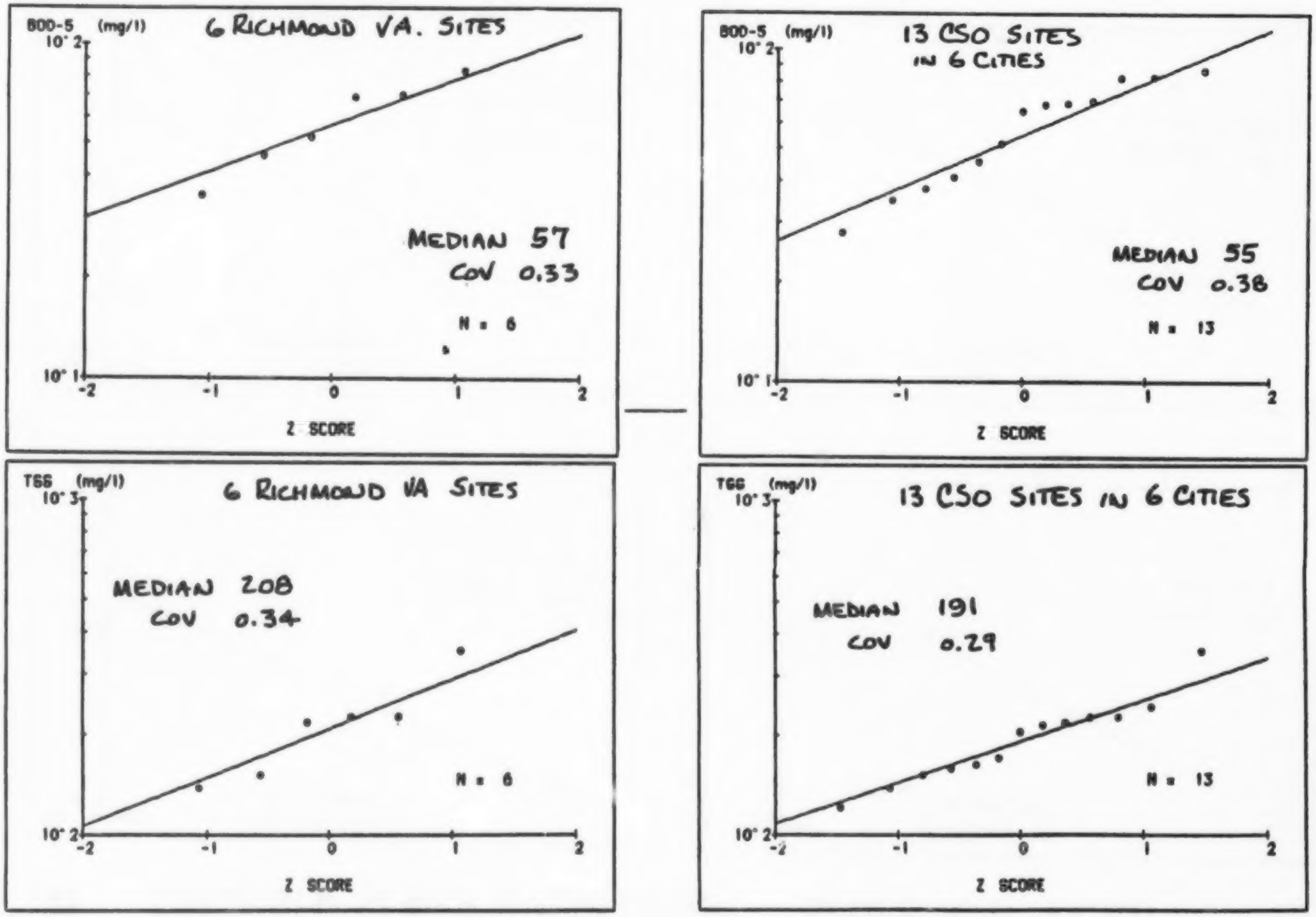






7

COMBINED SEWER OVERFLOWS - SITE MEDIANS

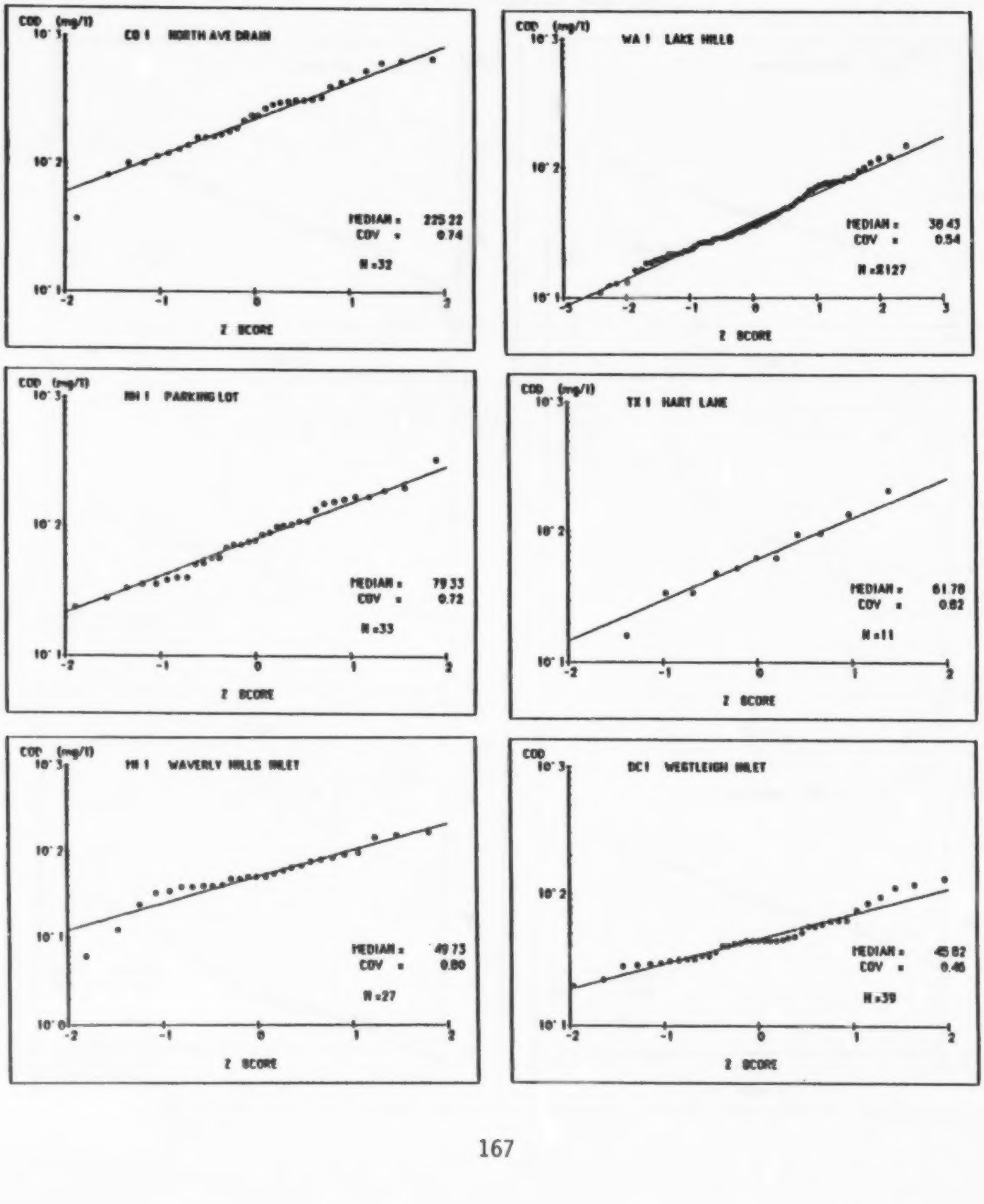


8

URBAN RUNOFF

DATA FROM NURP STUDY

EVENT MEAN CONCENTRATIONS OF CHEMICAL OXYGEN DEMAND
AT SIX STUDY SITES

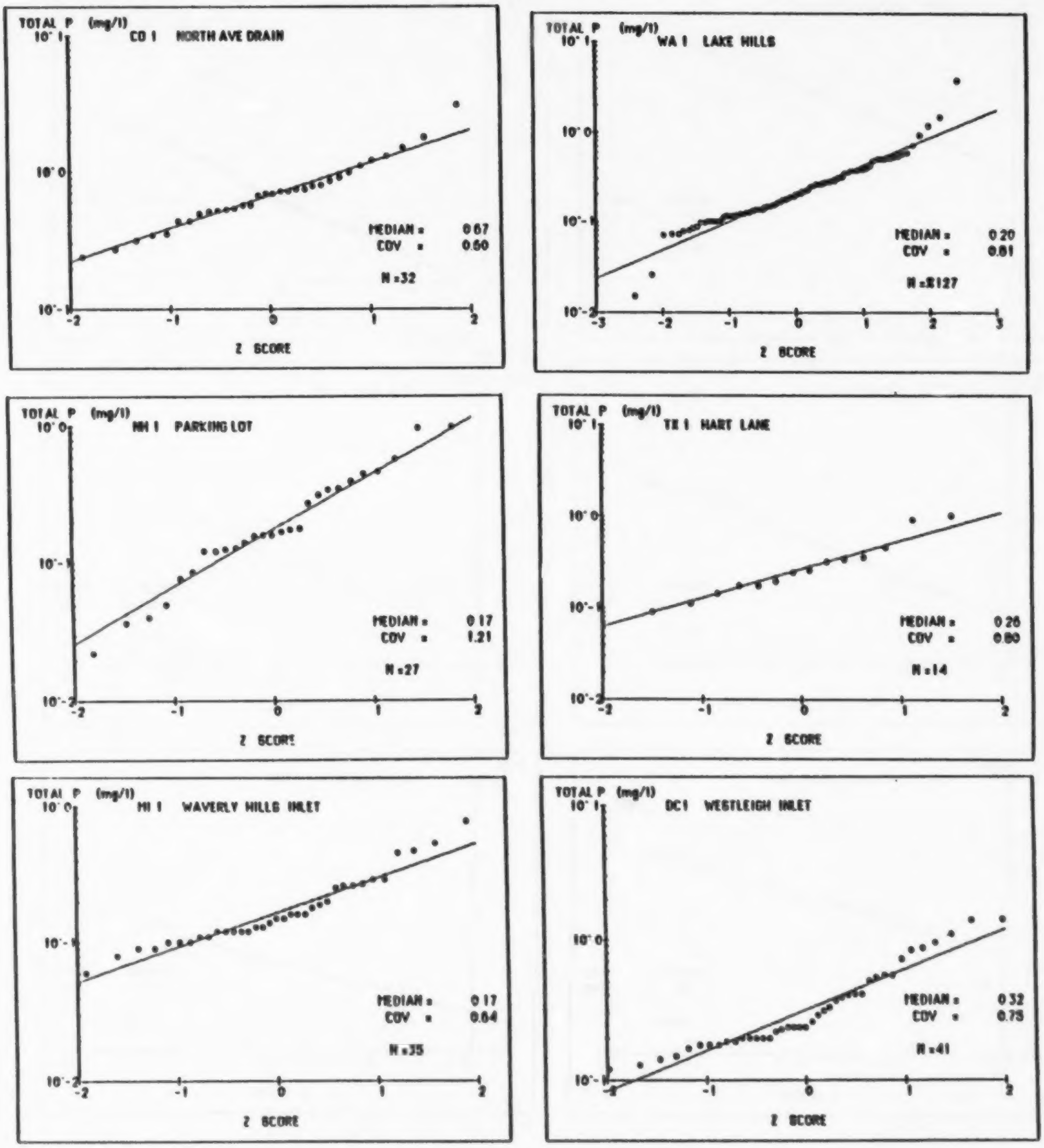


9

URBAN RUNOFF

DATA FROM NURP STUDY

EVENT MEAN CONCENTRATIONS OF TOTAL P AT SIX STUDY SITES



10

URBAN RUNOFF

DATA FROM NURP STUDY

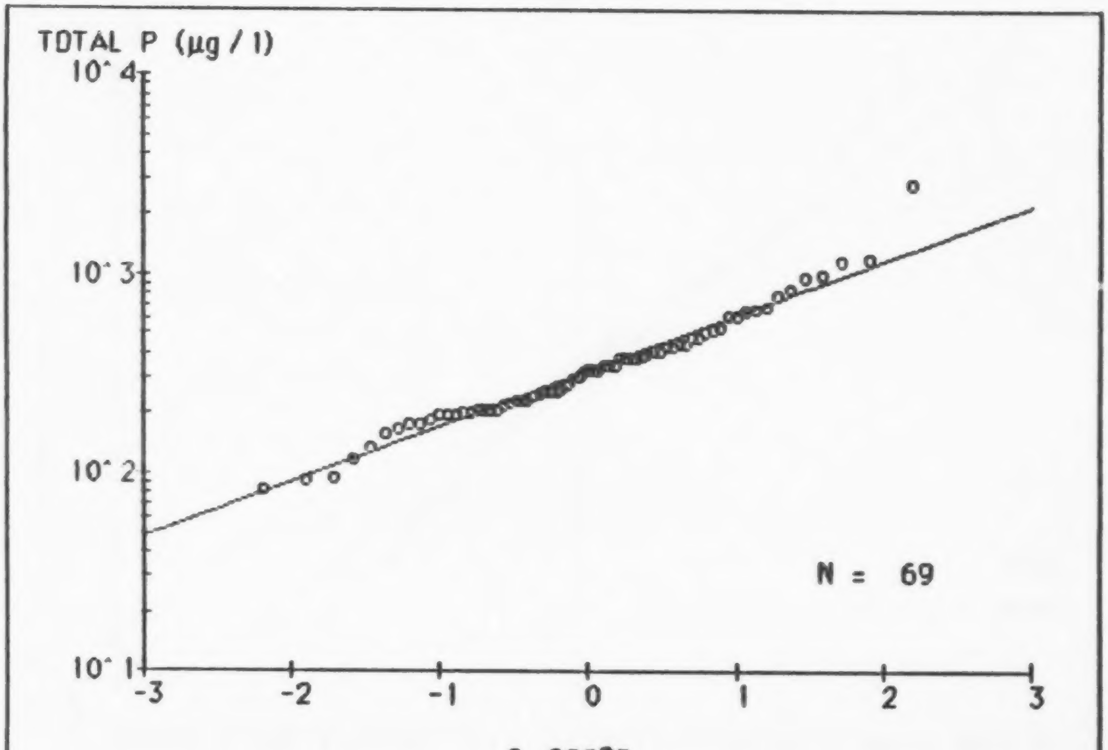
SITE MEDIAN CONCENTRATIONS OF TOTAL P

FOR 69 URBAN RUNOFF STUDY SITES

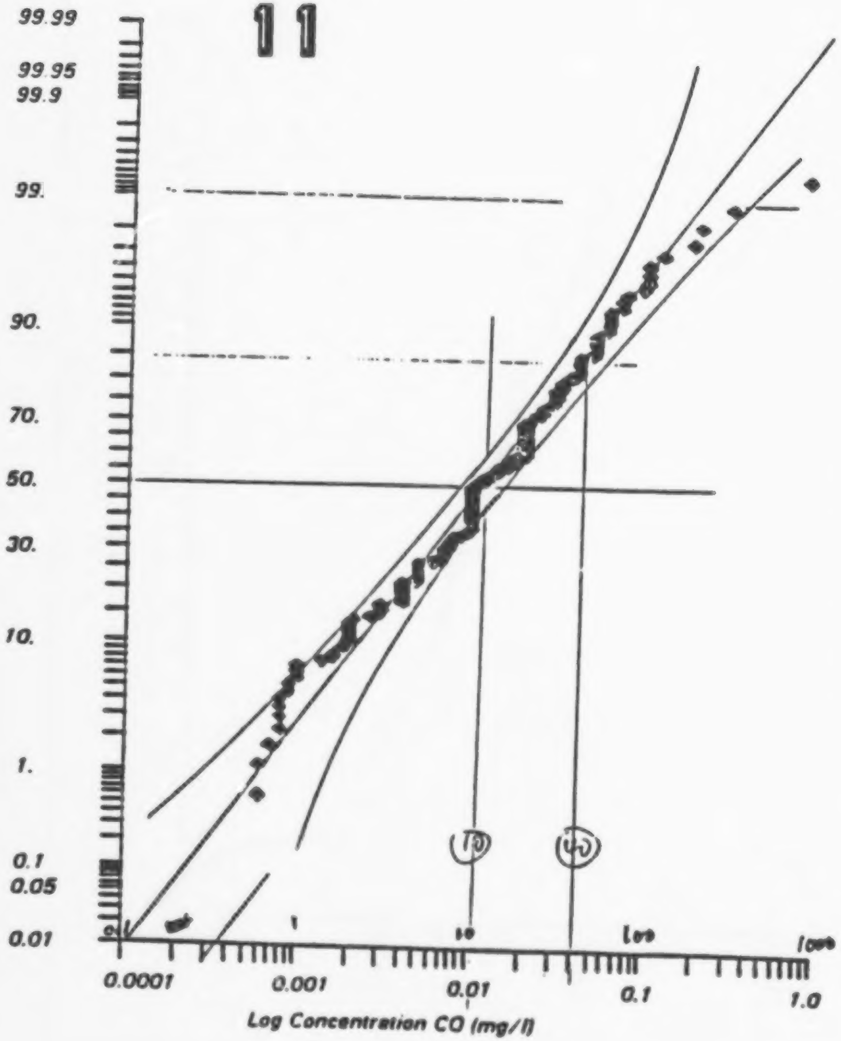
```
LOG MEAN  =      5.787
LOG SIGMA =      0.640

 MEAN     =    400.143
 SIGMA    =    284.565
 MEDIAN   =    326.092
COEF VAR  =      0.711
```

****



11



POTW - INFLUENT
MEDIAN
CADMIUM CONCENTRATIONS

DATA BASE FOR INFLUENT
HEAVY METAL CONCENTRATIONS
IN POTW's

EPA 600/S2-81-220

2 YEAR STUDY
DATA FROM 239 PLANTS

"Generally, log normal distributions were observed for all but the data extremes."

"Overall, the individual plant mean and median metal concentration data tend to fit a log normal distribution."

****

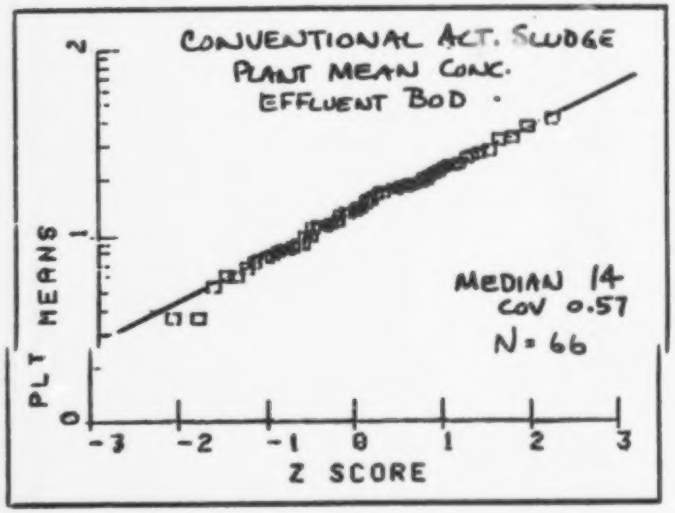
FIGURE SHOWS

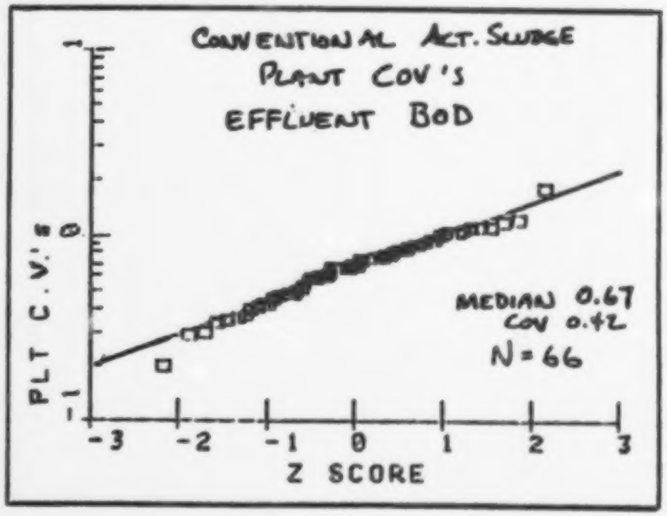
LOG PROBABILITY PLOT
for reported
MEDIAN CADMIUM CONCENTRATIONS

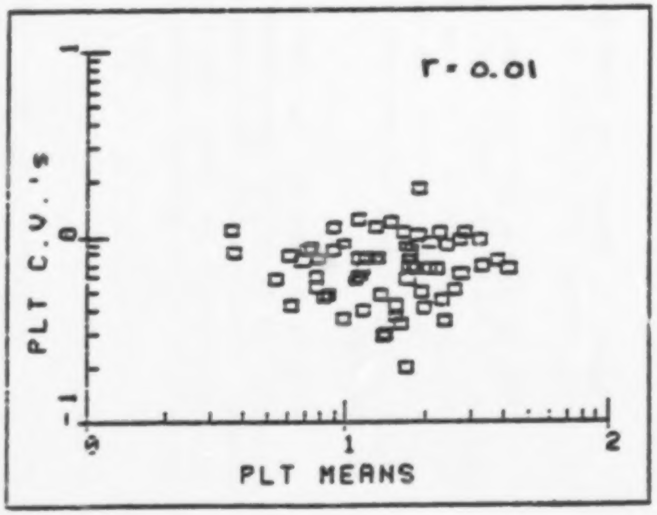
12

POTW

EFFLUENT CHARACTERISTICS
BOD







AGRICULTURAL RUNOFF

Figure 13 shows the distribution of the EMC's of three pollutants (ammonia, nitrate and TKN) at two study sites in Watkinsville, Georgia.

The ammonia nitrogen EMC's for agricultural runoff from four study sites in the Four Mile Creek watershed in Iowa are presented in Figure 14. Total Phosphorus EMC's at two of these sites are shown by Figure 15. The second of the plots for each site excludes 4 observations during one of the winter periods when manure was applied on frozen ground and not worked into the soil.

Figure 16 displays the distribution of annual average values for the concentration of soluble phosphorus and nitrogen in both surface runoff and subsurface stormwater discharges at the site of a long term study at a different location in Iowa. The final plots describe the variability of the concentration ratio between subsurface and surface discharges.

CONCLUSIONS

1. Whether or not it is rigorously true that the distribution of the underlying population of pollutant concentrations from the indicated sources ar log normal (as suggested by the material presented), it appears to be an acceptable approximation that will be adequate for many of the uses to which it may be put.

2. Lognormality can be assumed for the distribution of EMC's at a site, or for the distribution of mean or median concentrations from a number of different sites that are in the same category.

The work described in this paper was not funded by the U. S. Environmental Protection Agency and therefore the contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

13 AGRICULTURAL NPS RUNOFF

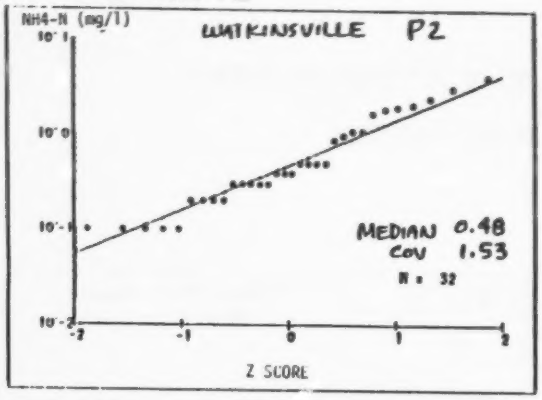
WATKINSVILLE GA STUDY

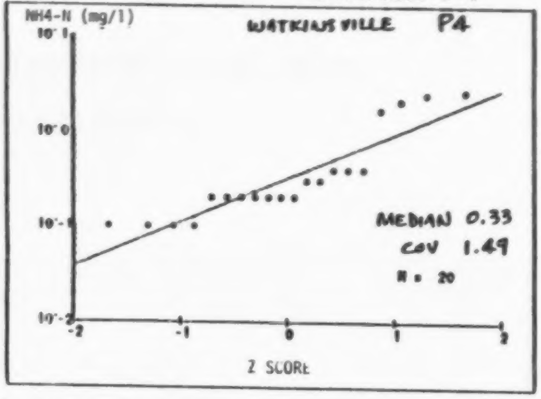
EVENT MEAN CONCENTRATIONS OF NITROGEN FORMS

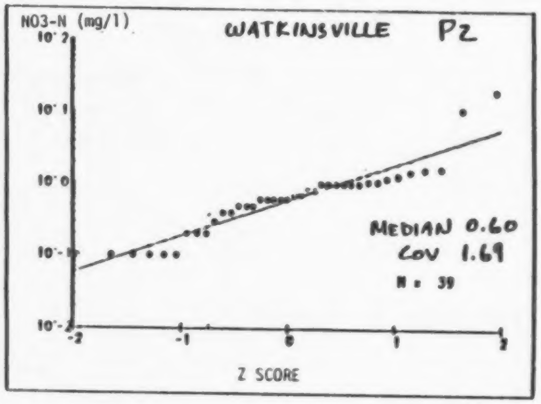
(NH_4 - N , NO_3 - N , TKN)

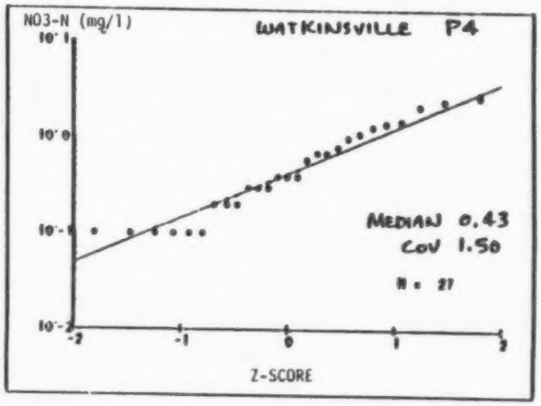
STATION P2

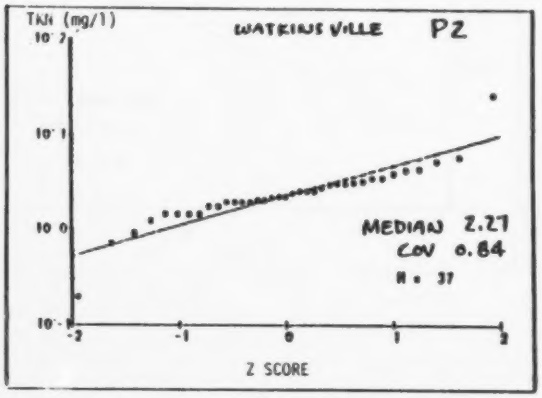
STATION P4

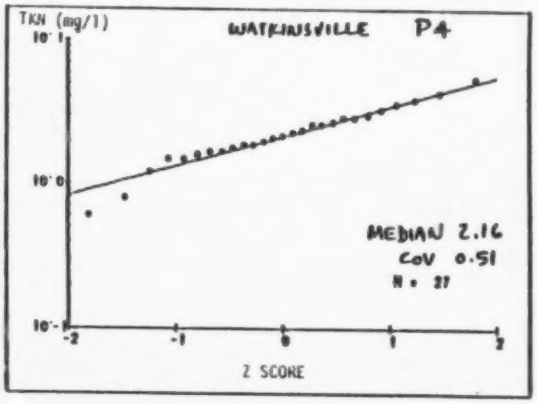










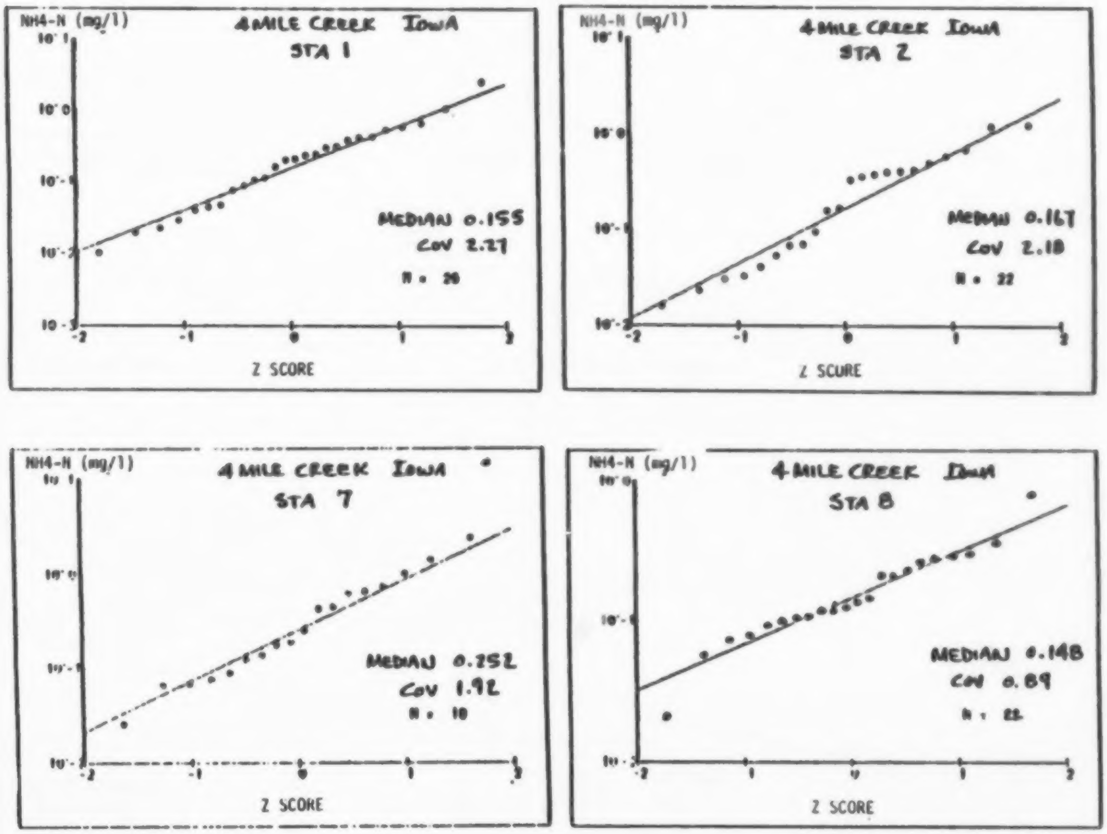


14 AGRICULTURAL NPS RUNOFF

FOUR MILE CREEK IOWA STUDY

EVENT MEAN CONCENTRATIONS OF AMMONIA NITROGEN

AT FOUR MONITORING STATIONS



15

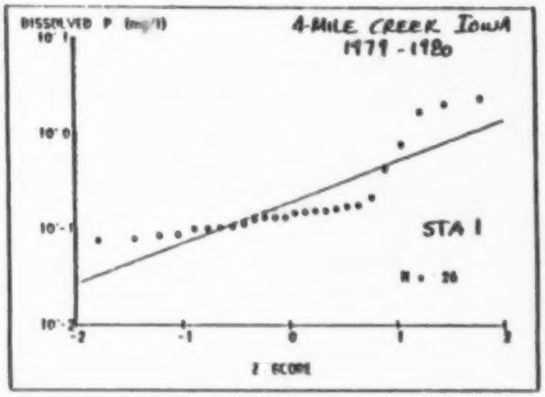
AGRICULTURAL NPS RUNOFF

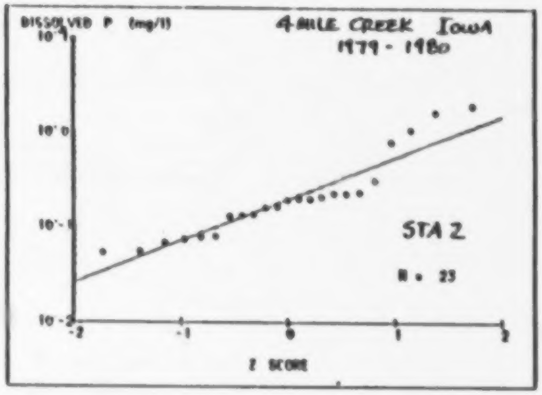
FOUR MILE CREEK IOWA STUDY

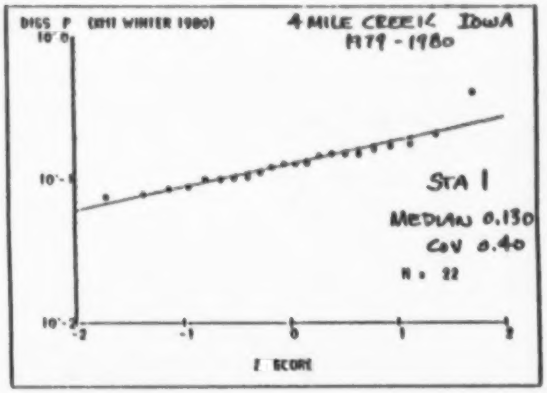
EVENT MEAN CONCENTRATIONS OF DISSOLVED PHOSPHORUS AT TWO MONITORING STATIONS

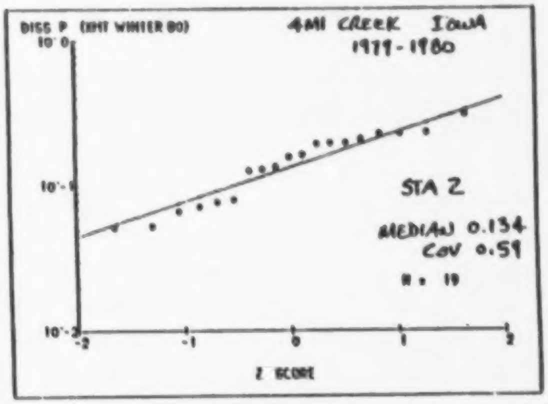
STATION 1

STATION 2







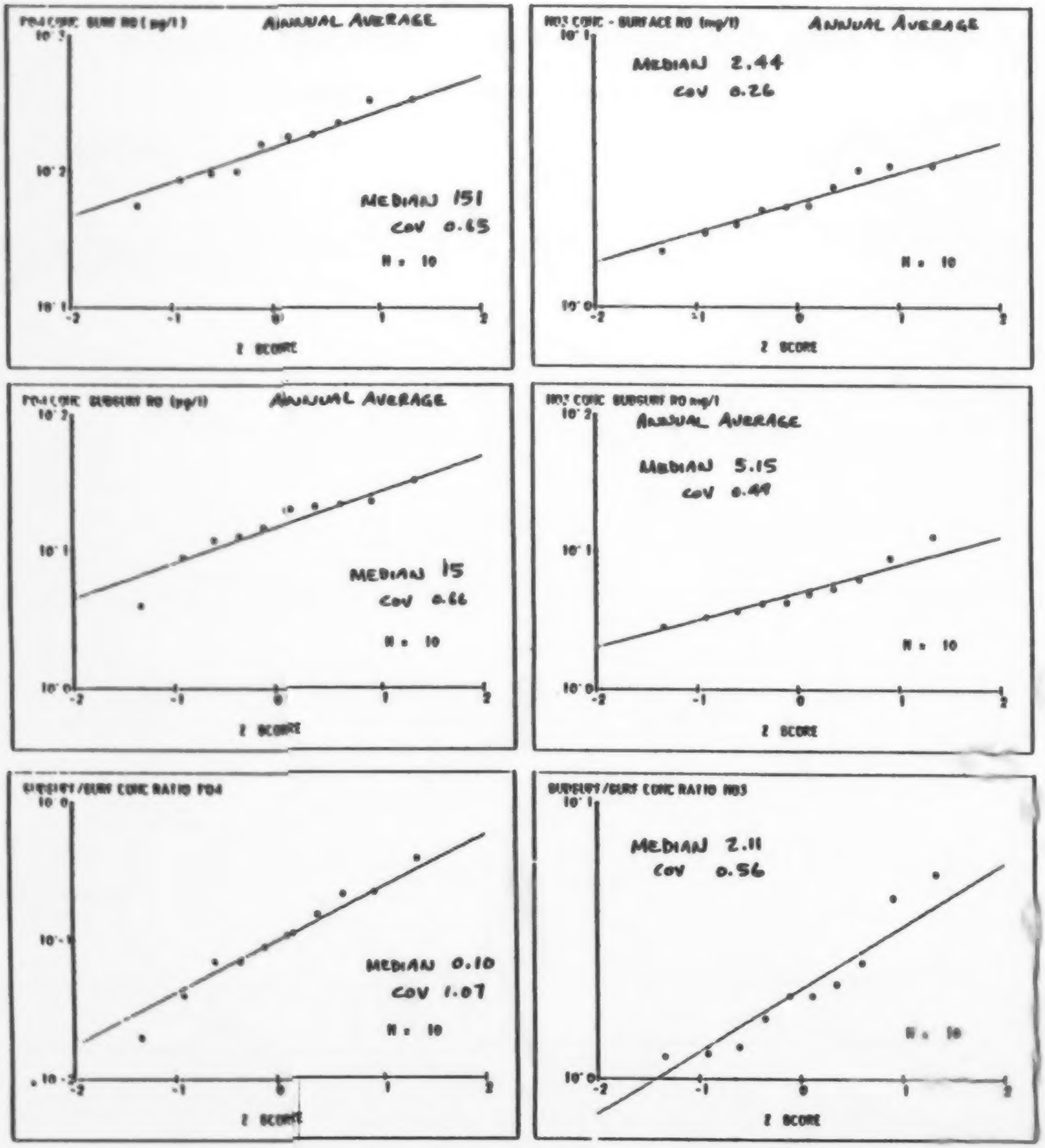


16 SURFACE AND SUBSURFACE RUNOFF CONVENTIONAL TILLAGE

ALBERTS and SPOONER, J SOIL AND WATER CONSERVATION, FEB 1985

ANNUAL AWERAGE CONCENTRATIONS OF SOLUBLE POLLUTANTS

PHOSPHORUS (PO_4) NITROGEN (NO_3)



POLLUTION FROM HIGHWAY RUNOFF—PRELIMINARY RESULTS

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ABSTRACT

This paper presents the preliminary results of a project aimed at developing models that can be used by planners and highway engineers for predicting pollutant runoff from highways. It is a part of an ongoing research program being conducted by the Federal Highway Administration of the U. S. Department of Transportation to characterize stormwater runoff from highways, assess its potential impacts on receiving waters, and determine the effectiveness of various control measures.

A brief review of different approaches to predicting pollutant runoff loads from highways is followed by a description of the data base being assembled as a part of this project. The probabilistic data analysis methodology that is being used to characterize highway stormwater runoff is then described in some detail. Preliminary analytical results are presented in two main areas—rainfall and runoff data and water quality data.

For the sites examined so far, it is shown that taking the percent imperviousness of an unmonitored site as its runoff coefficient, with an upper bound of around 0.9 to account for initial abstraction, offers a reasonable first estimate. Thus, the runoff quantity from an ungaged highway site can be estimated from a simple analysis of rainfall records from a gage near the site.

Based upon the their overall characteristics, the highway sites in the data base have been classified as being either "urban" or "rural." Best estimates for pollutant concentrations for each of six pollutants (SS, COD, TKN, TPO4, Pb, and Zn) for highway sites are presented and compared with corresponding values for urban runoff. Lacking site specific data, they represent reasonable first estimates of highway stormwater runoff quality. It is also shown that suspended solids can be a reasonable indicator for other pollutant concentrations in highway runoff.

INTRODUCTION

For over ten years the Federal Highway Administration (FHwA) of the United States Department of Transportation has been engaged in a research program to characterize stormwater runoff from highways, assess its impacts on receiving waters, and determine the effectiveness of various control measures for possible instances where their use to mitigate impairment of designated beneficial uses of receiving waters might be required. As a part of this research program, a contractor team headed by Woodward-Clyde Consultants, and including EG&G Washington Analytical Services Center, Inc. and the University of Florida, has been tasked with developing models that could be used by planners and highway engineers for predicting pollutant runoff loads from highways.

Approaches to predicting pollutant runoff loads from highways can be generally grouped into one of two classes;

- those that are based on regression equations, and
- those that use some form of simulation model.

Approaches that use simulation models can be further characterized mechanistically into three types:

- those employing rating curve methodologies—actually an extension of regression analysis based on a power law relationship between flow and sediment load or concentration;
- those where pollutant build-up and wash-off relationships are used as, for example, in SWMM, STORM, HSPF, and the FHwA Urban Highway Storm Drainage Model; and
- probabilistic based approaches, wherein probability density functions are assigned to runoff flows and concentrations, and the resulting pollutant loads are characterized statistically, as was done, for example, in the Environmental Protection Agency's Nationwide Urban Runoff Program.

Thus, prediction of highway stormwater runoff quality may be performed using a number of procedures that range from the very simple to detailed deterministic simulation models, and a large body of literature exists describing them. However, all of the approaches have one common aspect—their predictive capabilities tend to be rather poor without suitable site-specific data for calibration. For this reason, one of the first tasks in the present effort was to assemble a data base of highway runoff characteristics. In the following sections of this paper, that data base will be described, the data will be summarized, and best preliminary estimates of highway stormwater runoff will be made.

HIGHWAY RUNOFF DATA BASE CHARACTERISTICS

The data in the highway stormwater data base is taken from monitoring projects supported by the FHwA and several states. At the present time, there is coverage of twelve sites, with several more to be added as the data become available. There are from seven to 139 monitored events at each site. The data base consists of three categories of data;

- rainfall and runoff,
- water quality, and
- fixed site.

Each of these categories will be discussed in turn.

RAINFALL AND RUNOFF DATA

Rainfall data were taken with recording raingages. For each rainfall event there is a start time and date, five-minute raingage readings that describe the hyetograph, and a stop time and date. In a similar fashion, runoff data were taken from flowmeters and consist of a start time and date, (typically) five-minute instantaneous flow readings that describe the hydrograph, and a stop time and date. The lag between the time rainfall starts and the time runoff starts, although highly variable for a given site due to antecedent conditions, provides an indication of basin response time or "flashiness."

Taken together, the rainfall and runoff data describe the water quantity characteristics of the site. The rainfall data can be analyzed to provide information on rainfall frequency (i.e., the time between storms), intensity, duration, and total amount. For example, the total quantity of rainfall for the event is simply the difference between the raingage readings at the beginning and end of the event. The flowmeter data can be analyzed to determine peak flow, time to peak, total quantity discharged, etc. The total quantity of runoff discharged during the event is computed by integrating the flow rate record over the time period of the event. Of particular interest in this preliminary analysis are the total rainfall and runoff quantities.

WATER QUALITY DATA

The highway runoff quality data consist of the results of laboratory analyses of sequential discrete samples that were taken at recorded times throughout the runoff event, plus the results of laboratory analyses of flow-weighted composite samples taken over the entire runoff event.

Although over 24 analytical determinations were made for the different sites contained in the data base so far, there was consistent coverage of up to eighteen pollutants, and these were selected for this preliminary analysis. By category, they are:

• Solids	TS, TSS, VSS
• Physical Parameters	pH, Cl^-

• Oxygen Demanding Substances	BOD, COD, TOC
• Nutrients	TKN, NO_2+NO_3, TPO_4
• Metals (Total)	Cu, Pb, Zn, Fe, Cr, Cd
• Hydrocarbons	Oil and Grease

FIXED SITE DATA

The fixed site data, so called because they tend to be fixed rather than variable on an event to event basis, fall into three general categories. They are:

- Highway Site Data
  - configuration (e.g., elevated, ground level, depressed)
  - pavement composition, quantity, and condition
  - design, geometrics, cross-sections
  - vegetation types on right-of-way
  - drainage features
- Operational Characteristics
  - traffic characteristics (e.g., density, speed, braking)
  - vehicle characteristics (e.g., type, age, maintenance)
  - maintenance practices (e.g., sweeping, mowing, weed control)
  - institutional characteristics (e.g., litter laws, speed limit enforcement, vehicle emission regulations)
- Surrounding Land Use Characteristics
  - land use type (residential, commercial, industrial, agricultural, forest)
  - geologic features (relief, soil types and horizons, groundwater characteristics)
  - agricultural practices (e.g., tillage, irrigation, and cropping practices)

In this preliminary analysis, the chief fixed site data to be used are the size of the drainage area, the average traffic density expressed as vehicles per day, the percentage of the basin area that is impervious, and three roadway features—the surface type, number of lanes, and the presence of curbs.

DATA ANALYSIS METHODOLOGY

Pollutant runoff from highway sites (as well as from other types of nonpoint source sites) is quite variable, especially as compared to discharges from point sources. Therefore, new methodological frameworks are required for analysis of data from nonpoint sources in general and highway runoff in particular, especially with respect to characterization of pollutant loads and their possible effects upon receiving waters. New approaches to water quality criteria and standards also seem warranted when one is dealing with nonpoint sources, due to the high variability and intermittent nature of the latter.

When one has flow and polutant concentration data for runoff from a monitored site, there is a need for a sensible way to reduce the data to summary form in a way that will be useful to decision makers. For this task, it was decided to follow the general data analysis approach developed for the Environmental Protection Agency's Nationwide Urban Runoff Program (NURP).

In NURP, flow and concentration data for a site were first analyzed to determine event mean concentrations (EMCs). The EMC is defined as the concentration that would result if the entire storm event discharge were collected in a container, and its concentration determined; i.e., it is the total mass of pollutant discharged during the event divided by the total quantity of water discharged during the event. In practice the EMC is simply taken to be the concentration of a flow-weighted composite sample collected over the duration of the runoff event. If sequential discrete samples are taken over the period of the runoff event, an acceptable approximation to the EMC can be formed by manually compositing aliquots that are sized in proportion to flow (manual flow-weighted composite sample) and performing analytical determinations on the resulting composite sample. If separate analytical determinations have been performed on the individual sequential discrete samples, an acceptable approximation to the EMC often can be computed from the individual concentration values and the corresponding flow values.

The main point is that, in dealing with nonpoint sources, the basic unit of water quality information is the average concentration of each pollutant of interest in the total volume of runoff produced by each individual storm event. Thus, within-storm fluctuations in pollutant concentration are completely ignored. Hence, for a particular pollutant and a particular site, there will be one EMC value for each storm event monitored, and the EMC is considered to be the random variable.

Given a set of nonpoint source monitoring data for a site, the first step is to determine the EMC values for the pollutants of interest. This set of EMC values is then analyzed to compute the statistics of runoff quality for the site. However, in order to properly interpret the site statistics, it is usually convenient to know something about the nature of the underlying distribution function (or, equivalently, the underlying probability density function). From physical considerations, it can be shown that a normal distribution cannot be exactly correct for EMCs (e.g., pollutant concentrations can never be negative, they are always skewed, etc.). It can also be shown, by appeal to the central limit theorem, that when a series of multiplicative processes are involved (as is the case with nonpoint sources), the resulting distribution function will be asymptotically lognormal. The adequacy of this lognormality assumption has been checked with data from urban runoff, highway stormwater runoff, combined sewer overflows, and limited sets of agricultural runoff data. In no case has the lognormal distribution been clearly inappropriate, and in most cases it seems to fit the data better than other possible distributions.

Under the asumption that the lognormal distribution would be an adequate representation for the highway runoff data sets in question, it remained to compute the EMC values for each event at a site, transform these values into the logarithmic domain, compute the mean and variance in log-space, and then properly transform these values into the appropriate statistics in arithmetic space (typically the median, mean, and coefficient of variation). For surface runoff data, the two-parameter lognormal distribution is adequate. It is completely specified by a central tendency parameter (say, the median) and a dispersion parameter (say the coefficient of variation).

Having determined the two EMC statistics for a pollutant at a site (say, the median and coefficient of variation), they completely describe the variable runoff characteristics for that site. They can then be compared with similar data from other sites to evaluate similarities and differences in the context of physical site characteristics. This amounts to comparing the underlying probability distributions for the sites in question. The main point is that the foregoing methodology recognizes that the natural processes that lead to highway runoff are highly variable and provides an appropriate way of quantifying this variability.

Each data set is a small sample of the much larger population of values represented by the sample. The particular sample taken will be representative of the unknown population to an unknown extent, and may be fairly good or rather poor. There is no way to resolve this satisfactorily when one has only one sample available for inspection. However, when similar pollutant data sets are available for inspection from a large number of comparable sites, or for a variety of pollutants at the same site, the information to guide the desired inferences is extended. One way to shed further light on the representativeness of a nonpoint source data set is to compare the rainfall statistics for the monitored events with similar statistics computed from long-term rainfall records for the site. If the average rainfall volume for monitored storms is, for example, 0.80" while the long-term average for all storms is only 0.40", one can be reasonably certain that the monitored storms are not representative of the total population of storms at that site.

For reasons just alluded to, as well as the fact that an integral part of the assessment of the impact of storm loads on receiving water quality is the statistical evaluation of rainfall records, a program to summarize the important rainfall variables was set up on FHwA's computer as a part of this effort. The purpose of the Synoptic Rainfall Data Analysis Program (SYNOP) is to provide the user with a tool that can be used to summarize and statistically characterize a rainfall record in terms of its important variables (volume, intensity, duration, and time between storms). Since hourly rainfall records of many years duration are cumbersome and difficult to analyze, SYNOP provides an easy to use tool to facilitate the determination of, for example, seasonal trends important to the assesment of impacts and selection of control alternatives for storm-related loads.

SYNOP summarizes the hourly rainfall data by storm events, each with an associated volume (inches), duration (hours), average intensity (inches/hour), time since the previous storm (hours) as measured from the end of the previous storm, the antecedent rainfall (inches) for the last 24, 48, 72, and 168 hours, the hours of missing data, and the hours that the gage did not read. A storm definition, or interevent time, must be established to determine when, in the hourly record, a storm begins and ends. SYNOP delineates storm events as rainfall periods separated by a minimum number of consecutive hours without rainfall, a user supplied number. After the entire record has been read, SYNOP computes the statistics of relevant storm parameters by month and year and for the entire period of record.

PRELIMINARY RESULTS

RAINFALL AND RUNOFF DATA

Let us first consider the relationship of runoff to rainfall. Figure 1 is a scatter plot of the rainfall and runoff expressed in inches for a site on Highway Number 45 in Milwaukee, Wisconsin. There it can be seen that, although there is some scatter, there does tend to be a rather strong linear correlation, supporting the notion that a runoff coefficient might adequately characterize the site and allow runoff quantity to be predicted from rainfall data with a known degree of uncertainty. To examine this notion further, a scatter plot of runoff coefficient (Rv = runoff/rainfall) versus rainfall is presented in Figure 2. The lack of any correlation further supports the notion of a single runoff coefficient.

Although the claim for a single runoff coefficient is indicated in the foregoing, it is less than totally compelling. One of the reasons for the scatter might lie in the limited sample size for that site. We now turn to a site on Highway 94 in Milwaukee for which there are 137 monitored events. The corresponding scatter plots are presented as Figures 3 and 4. As can be seen, the argument becomes much more compelling.

A summary of the highway site rainfall and runoff data is presented in Table 1. The computed statistics (mean, median, coefficient of variation—the standard deviation divided by the mean and abbreviated as COV, and the number of samples N) are indicated for rainfall quantity, storm duration, runoff quantity, and runoff coefficient—the runoff divided by rainfall and abbreviated as Rv. The results of regression analyses of the form

$$\text{Runoff} = A \times \text{Rain} + B \qquad \text{and} \qquad Rv = A \times \text{Rain} + B$$

are presented in Table 2. In the first case, the value of A is the desired runoff coefficient, and one wants the residual term (B) to be small. In the second case, the value of B is the runoff coefficient, and one wants the value of A to be small. As can be seen from Table 2, the foregoing tends to be the case for all but the last two sites. The mean, median, and coefficient of variation for the runoff coefficient for each site are presented for comparison purposes in Table 2.

Since one would expect that the percent impervious area will affect the runoff coefficient for a site, this is examined in Figures 5 and 6. Taken together, these figures suggest that, lacking any other information, it is not unreasonable to assign a runoff coefficient equal to the percent imperviousness for a site, with an upper bound of around 0.9 or so to account for initial abstraction.

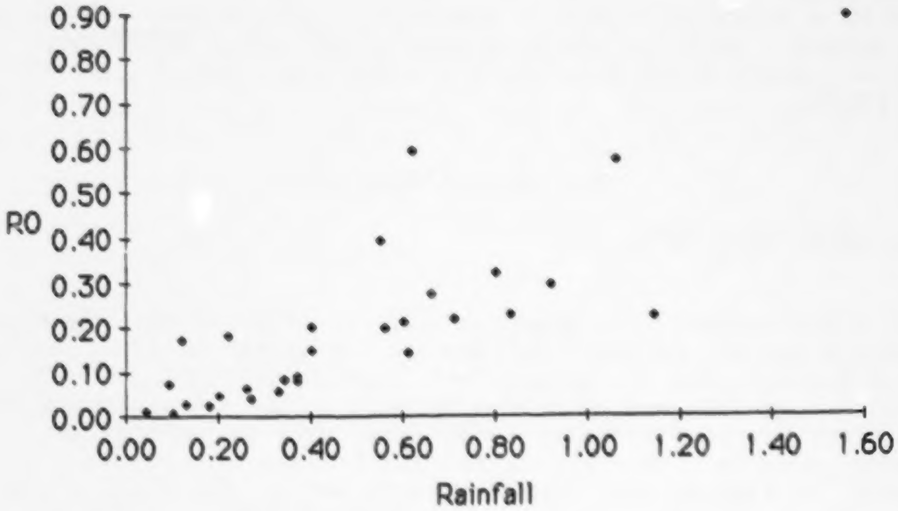

Figure 1. Runoff (RO) versus rainfall for Milwaukee Highway 45

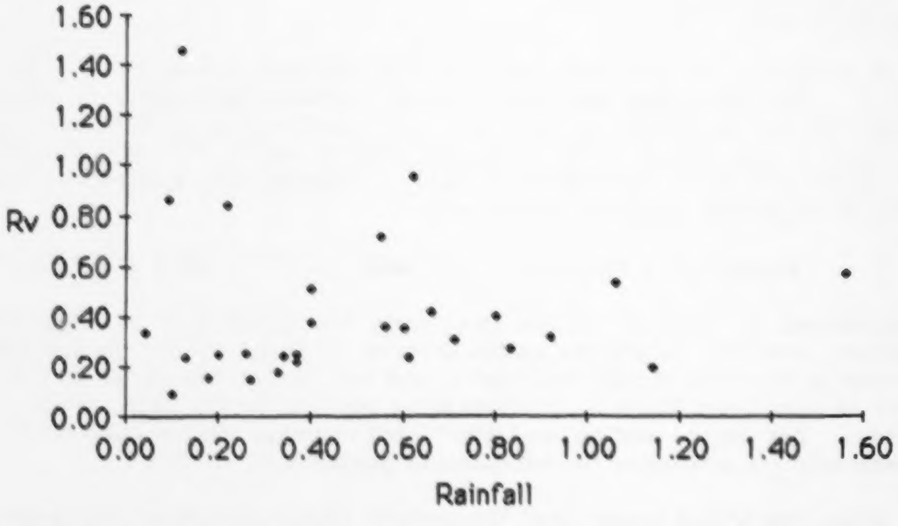

Figure 2. Runoff coefficient (Rv) versus rainfall for Milwaukee Highway 45

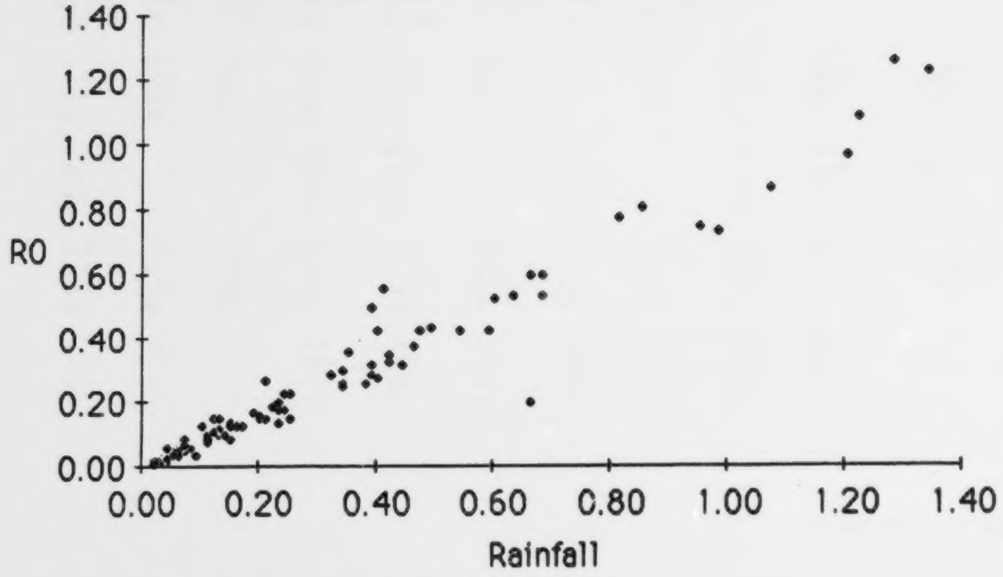

Figure 3. Runoff (RO) versus rainfall for Milwaukee Highway 94

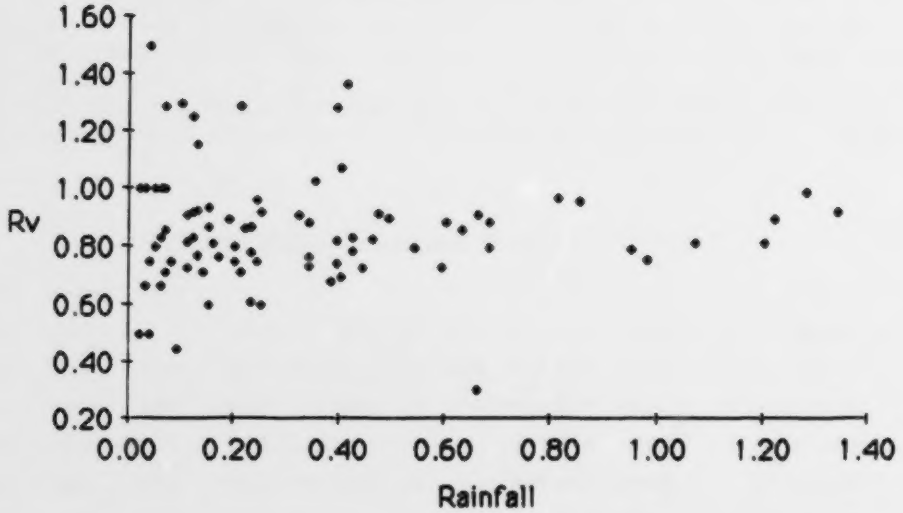

Figure 4. Runoff coefficient (Rv) versus rainfall for Milwaukee Highway 94

TABLE 1. SUMMARY HIGHWAY SITE RAINFALL/RUNOFF DATA

		Denver I-25	Milwaukee I-94	Nashville I-40	Sacramento Hwy. 50	Milwaukee Hwy. 45	Milwaukee I-795	Harrisburg I-81 (Ph. II)	Efland I-85	Harrisburg I-81 (Ph.I)
RAIN (in.)	Mean	0.42	0.27	0.83	0.52	0.54	0.63	0.71	0.86	0.76
	Median	0.27	0.14	0.44	0.32	0.37	0.39	0.49	0.66	0.61
	COV	1.14	1.64	1.58	1.31	1.05	1.26	1.07	0.84	0.74
	N	16	137	31	34	29	35	21	36	23
DURATION (hr)	Mean	5.23		5.34		7.29	5.56			8.14
	Median	1.93		2.76		3.34	3.28			4.35
	COV	2.51		1.66		1.94	1.36			1.58
	N	16		31		29	35			23
RUNOFF (in.)	Mean	0.194	0.24	0.34	0.45	0.23	0.53	0.18	0.84	0.40
	Median	0.086	0.11	0.16	0.26	0.13	0.32	0.02	0.33	0.18
	COV	2.01	1.89	1.97	1.43	1.53	1.30	8.63	2.33	2.03
	N	16	139	31	34	29	34	21	38	23
Rv	Mean	0.36	0.84	0.41	0.82	0.42	0.86	0.11	0.67	0.44
	Median	0.31	0.81	0.35	0.81	0.35	0.85	0.04	0.47	0.31
	COV	0.54	0.28	0.58	0.18	0.68	0.14	2.32	1.03	1.72
	N	16	137	31	34	29	35	21	36	23

RUNOFF QUALITY DATA

To compare the runoff data from the highway sites in the data base, data for six of the eighteen water quality parameters are presented in Table 3. Since data from NURP and other nonpoint source studies have indicated that data from individual sites tends to be lognormally distributed, the summary statistics given in Table 3 were computed in the same way as the summary statistics for an individual site. In order to provide a basis for comparison, the median values computed from the NURP data base are also presented in Table 3, along with the ratio of the highway median values to them. Thus, it can be seen that the concentrations of suspended solids and phosphorus in highway runoff are virtually the same as those in urban runoff, concentrations of COD and zinc are slightly higher in highway runoff, and concentrations of lead and TKN are significantly higher in highway runoff. In reviewing these numbers, it is important to remember that we

TABLE 2. RUNOFF/RAINFALL REGRESSIONS

			Mean	Median	COV
Milwaukee, I-94	Runoff= 0.86 RAIN - 0.00	Rv= 0.04 RAIN + 0.83	0.84	0.81	0.28
Milwaukee, I-795	Runoff= 0.84 RAIN + 0.01	Rv= 0.06 RAIN + 0.80	0.86	0.85	0.14
Sacramento, Hwy 50	Runoff= 0.88 RAIN - 0.02	Rv= 0.06 RAIN + 0.79	0.82	0.81	0.18
Milwaukee, Hwy 45	Runoff= 0.45 RAIN - 0.02	Rv= -0.02 RAIN + 0.43	0.42	0.35	0.68
Nashville, I-40	Runoff= 0.33 RAIN - 0.04	Rv= -0.02 RAIN + 0.41	0.42	0.35	0.58
Denver, I-25	Runoff= 0.60 RAIN - 0.06	Rv= 0.37 RAIN + 0.21	0.36	0.31	0.54
Harrisburg Ph II, I-81	Runoff= 0.29 RAIN - 0.08	Rv= 0.15 RAIN - 0.01	0.11	0.04	2.32
Efland, I-85	Runoff= 0.94 RAIN - 0.20	Rv= 0.20 RAIN + 0.41	0.67	0.47	1.03
Harrisburg Ph I, I-81	Runoff= 0.84 RAIN - 0.25	Rv= 0.39 RAIN + 0.12	0.44	0.31	1.72

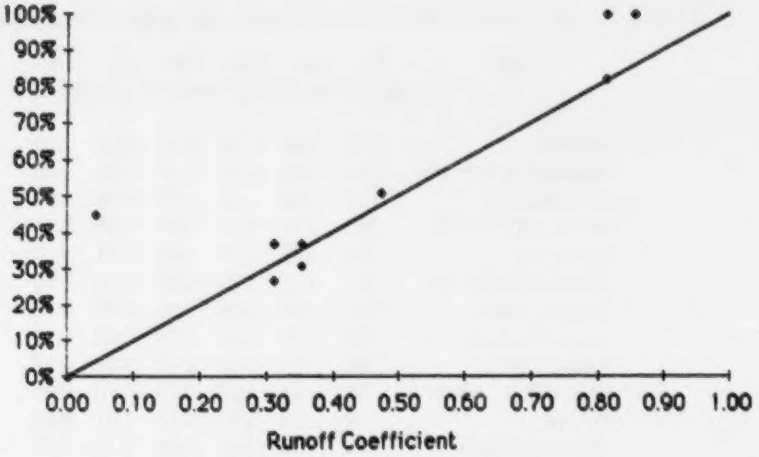

Figure 5. Percent imperviosness versus mean runoff coefficient

are talking about concentrations and not loads. On a load basis, highways would be much smaller contributers, because they account for only a small percentage of the land in an urban area.

Ratio techniques are frequently used to facilitate comparison among similar data sets, since they provide a simplistic form of normalization. In the present case we have divided the median value for a pollutant at a site by the median value for all of the sites to form the ratio. A value greater than unity indicates that that particular site is "dirtier" than average, while a value less than unity indicates that the site is "cleaner" than average. The results are presented in

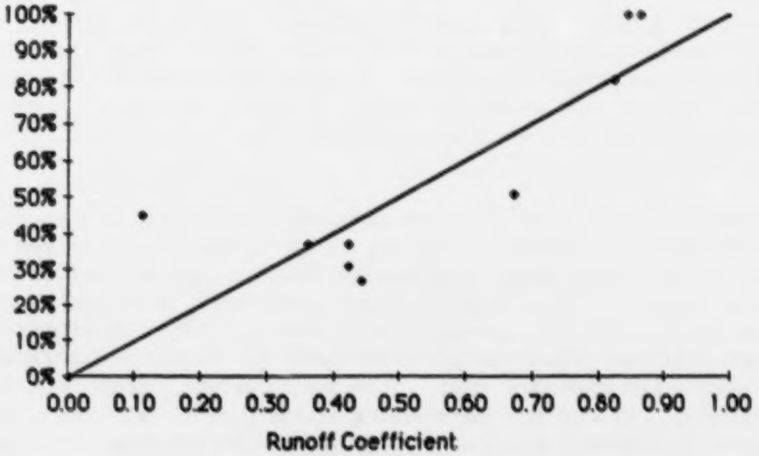

Figure 6. Percent imperviousness versus median runoff coefficient

TABLE 3. SITE MEDIAN DATA SUMMARY AND COMPARISON

SITE	SS (mg/l)	COD (mg/l)	TKN (mg/l)	TP04 (mg/l)	Pb (mg/l)	Zn (mg/l)
DENVER	410	289	3.38	0.82	0.68	0.62
MILWAUKEE HWY 795	183	130	2.52	0.38	2.03	0.46
LOS ANGELES	172	196	4.22	0.45	0.99	0.55
MILWAUKEE HWY 45	343	134	2.76	0.45	0.88	0.44
NASHVILLE	190	113	1.90	1.69	0.41	0.26
MILWAUKEE HWY 94	161	122	3.20	0.30	0.90	0.52
WALNUT CREEK	224	120	2.24	0.41	0.75	0.30
HARRISBURG (Ph. II)	184	34	2.20	1.08	0.03	0.17
SACRAMENTO	90	51	1.90	0.12	0.28	0.27
HARRISBURG (Ph. I)	31	31	1.20	0.29	0.10	0.06
EFLAND	19	49	2.50	0.13	0.01	0.06
BROWARD COUNTY	9	38	0.68	0.06	0.23	0.07
Mean	221	113	2.46	0.55	1.03	0.35
Median	108	86	2.18	0.35	0.31	0.24
COV	1.78	0.85	0.52	1.21	3.21	1.09
NURP MEDIAN	100	65	0.68	0.33	0.14	0.16
HIGHWAY/NURP	1.08	1.32	3.21	1.07	2.18	1.48

Table 4. The last column in Table 4 (labeled mean) is a sort of site cleanliness index, formed by simply averaging the ratio values for each of the six pollutants at the site.

Based upon data in the fixed site data base, we have chosen to simplistically group the highway sites into one of two categories, urban and rural, and repeat the ratio analysis just described. The results are presented in Tables 5 and 6. From these data, it appears that median pollutant concentrations in runoff from urban highway sites are from two to four times greater than those found in urban runoff, while those from rural highway sites tend to be around one quarter to a little over one half of that found in urban runoff. TKN is an exception to this last observation, and the data will be examined more carefully to attempt to find an explanation.

Many workers have tried to relate pollutant concentrations in runoff to suspended solids values, attempting to use the latter as an indicator parameter. We have done this on an event basis, and sample results for the Denver highway site are presented in Table 7. The relatively low coefficients of variation suggest that, for this site at least, such an approach has some validity. Summary results for all of the urban highway sites are given in Table 8. Only the coefficient of variation for lead seems surprisingly high. The two suspect values are those for the Milwaukee Highway 795 and the Harrisburg Phase II sites, and they will be examined critically in subsequent analyses. Similar results for the rural highway sites are given in Table 9.

TABLE 4. SITE MEDIAN RATIO SUMMARY AND COMPARISON

SITE	SS	COD	TKN	TP04	Pb	Zn	MEAN
DENVER	3.78	3.36	1.55	2.32	2.23	2.62	2.64
MILWAUKEE HWY 795	1.69	1.51	1.16	1.08	6.65	1.94	2.34
LOS ANGELES	1.59	2.28	1.94	1.27	3.24	2.32	2.11
MILWAUKEE HWY 45	3.17	1.55	1.27	1.26	2.87	1.86	2.00
NASHVILLE	1.75	1.31	0.87	4.77	1.34	1.10	1.86
MILWAUKEE HWY 94	1.49	1.42	1.47	0.85	2.95	2.20	1.73
WALNUT CREEK	2.07	1.40	1.03	1.16	2.46	1.27	1.56
HARRISBURG (Ph. II)	1.70	0.40	1.01	3.06	0.09	0.71	1.16
SACRAMENTO	0.83	0.59	0.87	0.34	0.92	1.14	0.78
HARRISBURG (Ph. I)	0.28	0.36	0.55	0.82	0.32	0.26	0.43
EFLAND	0.18	0.57	1.15	0.37	0.04	0.25	0.42
BROWARD COUNTY	0.08	0.44	0.31	0.17	0.75	0.30	0.34

TABLE 5. URBAN SITE MEDIAN DATA SUMMARY AND COMPARISON

SITE	SS (mg/l)	COD (mg/l)	TKN (mg/l)	TP04 (mg/l)	Pb (mg/l)	Zn (mg/l)
DENVER	410	289	3.38	0.82	0.68	0.62
MILWAUKEE HWY 795	183	130	2.52	0.38	2.03	0.46
LOS ANGELES	172	196	4.22	0.45	0.99	0.55
MILWAUKEE HWY 45	343	134	2.76	0.45	0.88	0.44
NASHVILLE	190	113	1.90	1.69	0.41	0.26
MILWAUKEE HWY 94	161	122	3.20	0.30	0.90	0.52
WALNUT CREEK	224	120	2.24	0.41	0.75	0.30
HARRISBURG (Ph. II)	184	34	2.20	1.08	0.03	0.17
Mean	234	149	2.81	0.70	1.31	0.42
Median	220	124	2.72	0.59	0.55	0.38
COV	0.36	0.67	0.27	0.66	2.14	0.47
NURP MEDIAN	100	65	0.68	0.33	0.14	0.16
HIGHWAY/NURP	2.20	1.91	4.00	1.78	3.94	2.40

TABLE 6. RURAL SITE MEDIAN SUMMARY AND COMPARISON

SITE	SS (mg/l)	COD (mg/l)	TKN (mg/l)	TP04 (mg/l)	Pb (mg/l)	Zn (mg/l)
SACRAMENTO	90	51	1.90	0.12	0.28	0.27
HARRISBURG (Ph. I)	31	31	1.20	0.29	0.10	0.06
EFLAND	19	49	2.50	0.13	0.01	0.06
BROWARD COUNTY	9	38	0.68	0.06	0.23	0.07
Mean	42	43	1.65	0.16	0.26	0.12
Median	26	41	1.40	0.13	0.09	0.09
COV	1.24	0.23	0.62	0.71	2.64	0.85
NURP MEDIAN	100	65	0.68	0.33	0.14	0.16
HIGHWAY/NURP	0.26	0.64	2.06	0.39	0.67	0.56

TABLE 7. DENVER SITE SOLIDS RATIO RESULTS

EVENT	SS/SS	COD/SS	100TKN/SS	100TPO4/SS	100Pb/SS	100Zn/SS
1	1.00	0.84	1.27	0.29	0.18	0.21
3	1.00	0.58	1.12	0.23	0.19	0.16
4	1.00	0.42	0.82	0.19	0.21	0.13
5	1.00	0.56	0.29	0.12	0.14	0.11
6	1.00	0.72	0.59	0.17	0.14	0.13
7	1.00	0.62	0.42	0.15	0.16	0.12
8	1.00	0.80	0.61	0.17	0.18	0.12
9	1.00	0.52	1.36	0.14	0.17	0.15
10	1.00	0.63	0.34	0.19	0.16	0.15
11	1.00	1.52	0.95	0.19	0.17	0.22
12	1.00	1.29	3.20	0.48	0.16	0.25
13	1.00	0.43	0.89	0.19	0.09	0.13
14	1.00	0.87	0.82	0.21	0.19	0.15
15	1.00	0.88	1.05	0.20	0.22	0.21
16	1.00	0.58	1.03	0.25	0.15	0.13
Mean	1.00	0.75	0.99	0.21	0.17	0.16
Median	1.00	0.70	0.82	0.20	0.16	0.15
COV	0.00	0.37	0.66	0.34	0.22	0.26
N	15	15	15	15	15	15

TABLE 8. URBAN SITE SOLIDS RATIO RESULTS

SITE	SS/SS	COD/SS	100TKN/SS	100TP04/SS	100Pb/SS	100Zn/SS
DENVER	1.00	0.70	0.82	0.20	0.17	0.15
MILWAUKEE HWY 795	1.00	0.71	1.38	0.21	1.11	0.25
LOS ANGELES	1.00	1.14	2.45	0.26	0.58	0.32
MILWAUKEE HWY 45	1.00	0.39	0.80	0.13	0.26	0.13
NASHVILLE	1.00	0.59	1.00	0.89	0.22	0.14
MILWAUKEE HWY 94	1.00	0.76	1.99	0.19	0.56	0.32
WALNUT CREEK	1.00	0.54	1.00	0.18	0.33	0.13
HARRISBURG (Ph. II)	1.00	0.18	1.20	0.59	0.01	0.09
Mean	1.00	0.65	1.34	0.33	0.59	0.19
Median	1.00	0.56	1.23	0.27	0.25	0.17
COV	0.00	0.59	0.42	0.73	2.15	0.50

TABLE 9. RURAL SITE SOLIDS RATIO RESULTS

SITE	SS/SS	COD/SS	100TKN/SS	100TP04/SS	100Pb/SS	100Zn/SS
SACRAMENTO	1.00	0.57	2.11	0.13	0.31	0.30
HARRISBURG (Ph. I)	1.00	1.02	1.33	0.32	0.11	0.07
EFLAND	1.00	2.58	2.78	0.14	0.01	0.06
BROWARD COUNTY	1.00	4.22	0.76	0.07	0.26	0.08
Mean	1.00	2.38	1.83	0.18	0.29	0.13
Median	1.00	1.58	1.56	0.14	0.10	0.10
COV	0.00	1.12	0.62	0.71	2.64	0.85

TABLE 10. URBAN SITE COEFFICIENT OF VARIATION RESULTS

SITE	SS	COD	TKN	TP04	Pb	Zn
DENVER	0.60	0.71	0.76	0.51	0.56	0.59
MILWAUKEE HWY 795	1.13	1.55	0.87	0.77	0.87	0.94
MILWAUKEE HWY 45	0.59	0.58	0.64	0.60	0.62	0.51
NASHVILLE	0.57	0.60	1.02	0.56	0.90	0.46
MILWAUKEE HWY 94	1.04	0.74	0.50	0.63	1.30	0.84
HARRISBURG (Ph. II)	1.16	0.40	0.51	0.52	3.93	0.51
Mean	0.86	0.77	0.72	0.60	1.36	0.65
Median	0.81	0.70	0.69	0.59	1.06	0.62
COV	0.36	0.47	0.29	0.15	0.81	0.30

Thus far we have only been dealing with pollutant concentration median values. The same types of analyses could be performed on the site coefficients of variation, and an example for the urban highway sites is given in Table 10. Here again, the lead values for the Harrisburg Phase II site seem suspect and warrant further examination. Otherwise, the results seem quite reasonable and fairly consistent.

The "bottom line" of this preliminary data analysis is presented in Table 11. If a planner has to estimate the pollutant load coming from an ungaged highway site, the values for concentration and coefficient of variation for each pollutant that are given there represent the best initial estimates for screening purposes.

As our work continues, we plan to refine the foregoing analyses and to add the remaining twelve pollutants as well as other highway sites. We also plan a rather extensive data quality assurance and quality control effort (unscreened raw data have been used up to this point). Hopefully, this will reduce the number of seeming anomalies pointed out earlier.

TABLE 11. HIGHWAY RUNOFF CHARACTERISTICS

		URBAN SITES	RURAL SITES	ALL SITES	COEFFICIENT OF VARIATION
SS	(mg/l)	220	26	108	0.8 – 1.0
COD	(mg/l)	124	41	86	0.5 – 0.8
TKN	(mg/l)	2.72	1.4	2.18	0.7 – 0.9
TP04	(mg/l)	0.59	0.13	0.35	0.6 – 0.9
Pb	(mg/l)	0.55	0.09	0.31	0.7 – 1.4
Zn	(mg/l)	0.38	0.09	0.24	0.6 – 0.7

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EFFECTIVENESS OF DETENTION/RETENTION BASINS FOR REMOVAL OF HEAVY METALS IN HIGHWAY RUNOFF

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ABSTRACT

The movement and fate of heavy metal inputs (Cd, Zn, Mn, Cu, Al, Fe, Pb, Ni and Cr) from highway runoff were investigated in a three-year study on a 1.3 hectare retention facility near the Maitland Interchange on Interstate 4, north of Orlando, Florida. Stormwater characteristics were compared with average retention pond water quality to determine removal efficiencies for heavy metals within the pond. A total of 138 sediment core samples were collected in the pond over a three-year period to investigate the horizontal and vertical migrations of heavy metals within the pond. Core samples were also carried through a series of sequential extraction procedures to examine the type of chemical associations and stability of each metal in the sediments. An apparatus was built which allowed sediments to be incubated under various conditions of redox potential and pH to investigate the effects of changes in sediment conditions on the stability of metal-sediment associations.

INTRODUCTION

Within the past decade, a substantial amount of research has accumulated relating to the water pollution caused by the operation of motor vehicles. This concern is based largely on the potential aquatic toxicity of heavy metals such as lead, zinc, and chromium. Heavy metals have been proposed by several researchers as the major toxicant present in highway runoff samples (Shaheen, 1975; Winters and Gidley, 1980). Many heavy metals are known to be toxic in high concentrations to a wide variety of aquatic plants and animals (Wilber and Hunter, 1977).

On a nationwide basis, the two most commonly used techniques for management of highway runoff are roadside swales and detention/retention facilities. As these facilities receive continual inputs of stormwater containing heavy metals, processes such as precipitation, coagulation, settling, and biological uptake will result in a large percentage of the input mass being deposited into the sediments. However, no previous definitive studies have been conducted to determine the fate of toxic species, especially heavy metals, in these stormwater management systems. In particular, no studies have been conducted to investigate if physical and chemical changes which may occur in these systems over time may mobilize certain species from the sediment phase back into the water phase.

The purpose of this research was to investigate the fate of heavy metals within stormwater management systems. The site selected for these investigations was a series of stormwater management facilities located at the Maitland Interchange on Interstate 4 north of the city of Orlando, Florida. A retention pond (West Pond) with relatively defined inputs and outputs was chosen as the primary study site.

SITE DESCRIPTION

The site selected for this investigation is the Maitland Interchange on Interstate 4. This interchange, located north of the city of Orlando, was constructed in 1976 (Figure 1). Three borrow pits dug to provide fill for constructing the overpass remain to serve as stormwater detention/retention facilities. The ponds are interconnected by large culverts and the northwestern (Pond B) has the capability to discharge to the southwestern (referred to hereafter as the West Pond) when design elevations are exceeded. However, under normal conditions, the only input into the West Pond is by way of a 45 cm concrete culvert that drains much of the Maitland Boulevard overpass. Discharge from the West Pond travels to Lake Lucien through a large culvert. A flashboard riser system regulates the water level in the West Pond, and a discharge rarely occurs to Lake Lucien.

The West Pond has an approximate surface area of 1.3 ha and an average depth of 1.5 m. The pond maintains a large standing crop of filamentous algae, particularly Chara, virtually year round. Because of the shallow water depth and large amount of algal production, the pond waters remain in a well oxygenated state. The sediment material is predominately sand which is covered by a 1 cm layer of organic matter.

Maitland Boulevard crosses over Interstate 4 by means of a bridge overpass created during construction of the Interchange. The Maitland Boulevard bridge consists of two sections, one carrying two lanes of eastbound traffic and an exit lane, the other carrying two lanes of westbound traffic and another exit lane. Traffic volume on Maitland Boulevard is approximately 12,000 average daily traffic (ADT) eastbound and 11,000 westbound. Traffic volume on I-4 through the Maitland Interchange is approximately 42,000 ADT eastbound and westbound each.

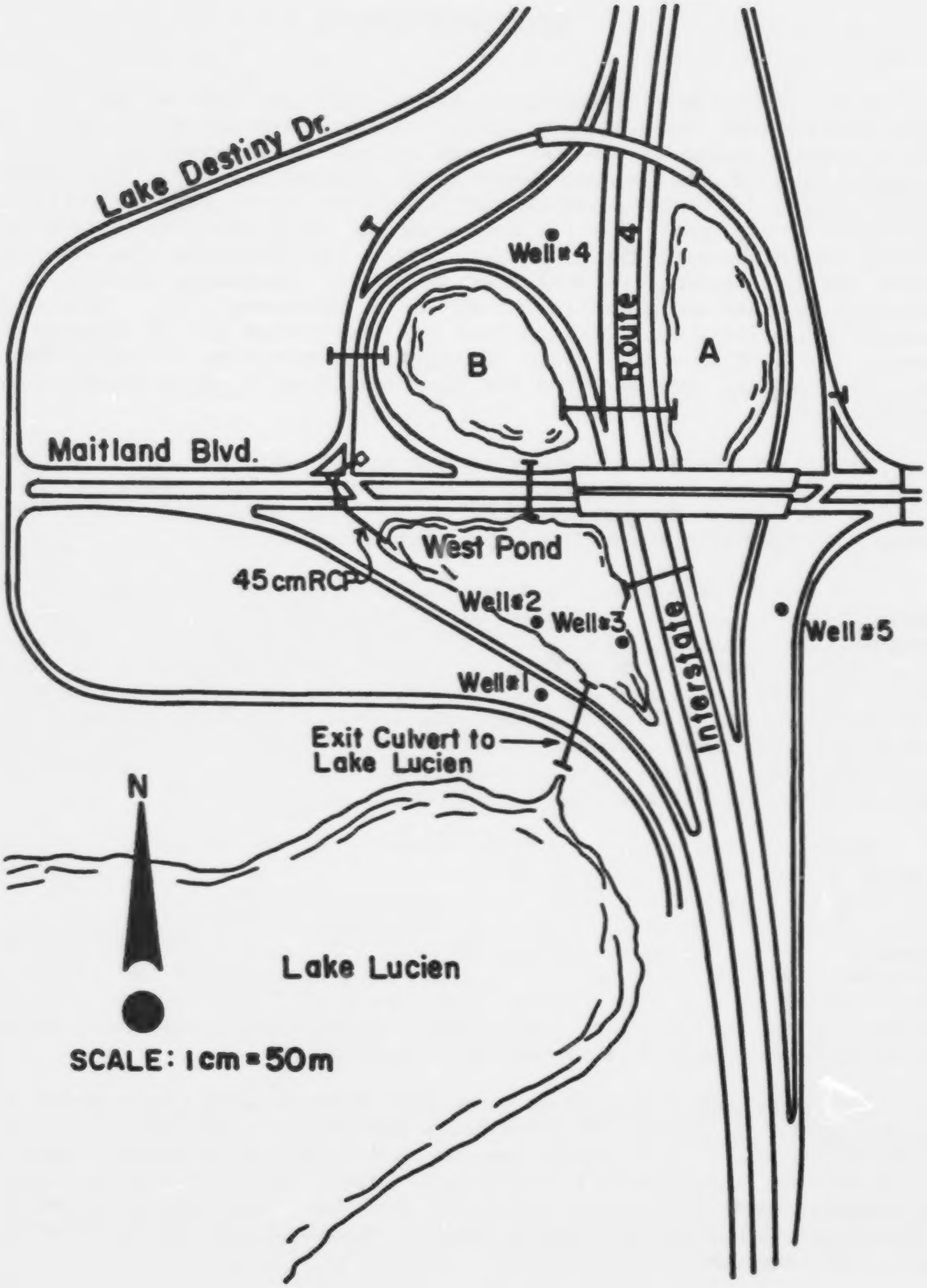
Lake Destiny Dr.
Well#4
Route 4
B
A
Maitland Blvd.
West Pond
45cmRCP
Well#2
Well#3
Well#5
Well#1
Interstate
Exit Culvert to
Lake Lucien
N
Lake Lucien
SCALE: 1cm=50m

Figure 1. Study Site at Maitland Interchange.

FIELD INVESTIGATIONS

Field investigations conducted during 1982 and 1983 at the West Pond were divided into the following tasks: 1) determination of the quantity of heavy metals entering the West Pond by way of stormwater runoff; (2) determination of the average heavy metal concentrations in the retention basin water; (3) assessment of the accumulation of heavy metals in the sediments of the pond; and (4) monitoring of heavy metal concentrations in ground waters beneath the retention basin. To determine the quantity of heavy metals entering the West Pond by way of stormwater runoff, an Isco automatic sampler was installed on the 45 cm stormsewer line. Flow-weighted composite samples were collected over a 1 year period for 16 separate storm events representing a wide range of rainfall intensities and antecedent dry periods. Samples were analyzed for heavy metals using argon plasma emission spectroscopy.

Water samples were collected on a biweekly basis for 1 year in the West Pond to document average retention pond water quality. Each of the five samples was analyzed separately for the heavy metals listed, and an average value was calculated for each metal on each sampling date.

The accumulation and vertical distribution of heavy metals in the sediments was examined by collection of a series of 2.5 cm diameter core samples to a depth of 15 cm. Forty-three separate core samples were collected in the 1.3 ha West Pond, and metal concentrations in sediment layers 0-1 cm, 1-3.5 cm, 3.5-6.0 cm, 6.0-8.5 cm and 8.5-13.0 were measured for each core sample. Metal concentrations in the 0-1 cm layer were used to investigate horizontal movement of heavy metals from the point of discharge into the pond. Average metal concentrations in each of the sediment layers were used to determine the extent of vertical migration.

Multiport monitoring wells were installed at the locations indicated in Figure 1 to investigate the possibility of groundwater contamination by leaching of heavy metals from the stormwater management system. Two of the monitoring wells were installed at the edges of the West Pond with the remaining three installed at various locations surrounding the stormwater management system. The wells were designed so that all of the sample ports were housed in a single casing to minimize soil disturbance and reduce recovery time for obtaining representative groundwater samples compared to other monitoring well designs such as cluster wells.

All wells were installed to a depth of 6 meters with sample ports at 0.1 m, 0.5 m, 1.0 m, 3.0 m, and 6.0 m below the average water table depth in the area of the well. Groundwater samples were collected from each sample port on a monthly basis using a peristaltic pump. Approximately 10 liters of groundwater were pumped and discarded from each port before a sample was collected. Collected groundwater samples were analyzed for heavy metals as described previously.

CHARACTERISTICS OF HIGHWAY RUNOFF AT THE MAITLAND SITE

A total of 16 storm events, including a total of 150 separate runoff samples were collected and analyzed over this period for dissolved and total heavy metals. Total rainfall amounts for sampled storm events ranged from 0.33 to 3.23 inches with antecedent dry periods of 0.24 to 25.4 days. Average flow rates in the 45 cm stormsewer, an indirect measure of rainfall intensity, ranged from 0.085 to 59.4 liters/sec.

A summary of mean flow-weighted heavy metal concentrations measured at the Maitland site is given in Table 1. Measured concentrations of heavy metals in highway runoff collected at the Maitland Interchange during 1983-84 showed considerable variability between storm events as well as during storm events. Average dissolved concentrations of all heavy metals, with the exceptions of iron and aluminum, were less than 70 ug/l, with nickel, chromium, manganese, and cadmium less than 3 ug/l. Measured mean concentrations of total metal species were in excess of 100 ug/l for lead, aluminum and iron, while nickel, chromium, manganese, and cadmium were all present in average total metal concentrations of 10 ug/l or less. In general, the variability of mean flow-weighted dissolved metal concentrations appears to be much less than that observed for mean total concentrations, with most dissolved species exhibiting a five-fold difference in range of concentrations, while total concentrations exhibited over a ten-fold range in most cases.

Of the heavy metals which were measured, the following orders were observed for mean concentrations of dissolved and total metal species:

Dissolved: Al > Fe > Zn > Pb > Cu > Mn > Ni = Cr > Cd
Total: Al > Fe > Pb > Zn > Cu > Mn > Cr > Ni > Cd

However, the metal species aluminum, iron and manganese are common constitutents of soils and may not be correlated with vehicle usage and highway operation, as would be expected for lead, nickel, chromium, copper, zinc, and cadmium. Therefore, the most common vehicle related heavy metals found in highway runoff at the Maitland site were lead, zinc, and copper in ratios of 4.70:1.91:1.0, respectively, for total concentrations, and ratios of 0.85:1.04:1.0, respectively, for dissolved species. Together these three metals accounted for approximately 91 percent of the dissolved heavy metals present and 94 percent of the total metal concentrations, excluding aluminum, iron and manganese.

With the exceptions of lead, iron, manganese, and aluminum, all heavy metals in the highway runoff samples appeared to be present predominantly in a dissolved form. Cadmium, nickel, and copper were all present in dissolved fractions which were near 75 percent of the total metal measured. On the other extreme, lead, iron, manganese, and aluminum were predominantly particulate in nature with dissolved fractions of only approximately 20 percent. Zinc and chromium appeared to be approximately equal in dissolved and particulate forms.

TABLE 1. HEAVY METAL CONCENTRATIONS IN SEQUENTIAL HIGHWAY RUNOFF SAMPLES COLLECTED AT THE MAITLAND WEST POND DURING 1983-84

HEAVY METAL	METAL SPECIES	NUMBER OF SAMPLES	MEAN (ug/1)	STANDARD DEVIATION	RANGE OF VALUES (ug/1)	PERCENT DISSOLVED (%)
LEAD	Dissolved	150	33.0	40.2	7.0 - 413	
	Total	150	181	331	11.0 - 3,596	18.2
ZINC	Dissolved	150	40.0	42.6	1.0 - 324	
	Total	150	73.9	71.2	5.0 - 372	54.1
COPPER	Dissolved	150	28.6	24.7	6.0 - 175	
	Total	150	38.6	28.8	6.0 - 176	74.1
NICKEL	Dissolved	150	2.5	2.4	0.5 - 15	
	Total	150	3.4	2.8	0.5 - 18	73.5
CHROMIUM	Dissolved	150	2.5	2.2	0.5 - 16	
	Total	150	4.2	3.2	0.5 - 18	59.5
IRON	Dissolved	150	77.9	59.7	11.0 - 466	
	Total	150	378	354	44.0 - 2,172	20.6
ALUMINUM	Dissolved	150	125	124	19.0 - 832	
	Total	150	561	563	53.0 - 3,499	22.3
MANGANESE	Dissolved	150	2.70	6.7	<1 - 59	
	Total	150	9.53	10.8	<1 - 62	28.3
CADMIUM	Dissolved	150	1.7	2.0	<1 - 12	
	Total	150	2.2	2.4	<1 - 12	77.3
pH		101	6.30	0.88	4.95 - 8.49	---

Probability distributions of mean flow-weighted heavy metal concentrations in the 16 measured storm events were examined by plotting flow-weighted mean metal concentrations for each measured event on probability paper. Total concentrations of heavy metals appeared to best approximate a straight line relationship with a log-normal distribution of concentrations for the events measured. Dissolved concentrations of heavy metals also seem to approximate a log-normal distribution. However, cadmium appears to exhibit a convex curvilinear relationship.

A "first flush" effect was observed for total concentrations of lead, zinc, iron, and aluminum. In general, approximately 50 percent of the total mass of these metals was found to be transported during the first quarter of a storm event, 25 percent during the second quarter, and the remaining 25 percent divided between the third and fourth quarters. This trend was not observed for total concentrations of the other metal species or for dissolved species of any measured metals.

FATE OF HEAVY METALS ENTERING THE MAITLAND POND

Although stormwater inputs into the pond were characterized by a considerable degree of variability, heavy metal concentrations measured in the pond were, in general, relatively consistent and low in value. Dissolved concentrations of all metal species in the pond with the general exception of aluminum, were never found to exceed 50 ug/l with dissolved concentrations of cadmium, zinc, manganese, nickel, and chromium rarely exceeding 10 ug/l. Total metal concentrations followed a similar pattern with only manganese, aluminum, iron, and on one occasion lead, exceeding 100 ug/l on any given sample day at any of the five sampling stations within the retention pond. The mean pH value of the retention pond water was 7.46 with a range of 6.62-8.46. Measurements of dissolved oxygen indicated an aerobic water column on all sample dates. The mean value for dissolved oxygen was 5.6 mg/l with measurements of ORP generally in excess of 500 mv.

The Maitland pond was found to be very effective in removal of heavy metal inputs from highway runoff. A comparison of summary statistics for highway runoff and Maitland pond water is given in Table 2. Heavy metal concentrations in this table have been given in terms of the concentrations of dissolved and particulate species rather than dissolved and total species. This was done so that the removal characteristics of soluble and particulate species could be examined separately.

Upon entering the Maitland retention pond, chemical, physical, and biological processes begin to occur which, for most metal species, results in substantial reductions in concentrations. The most noticeable removals for heavy metals occurs for the particulate species. Particulate species of lead and zinc are reduced in excess of 95 percent, cadmium and iron near 85 percent, with copper and aluminum averaging near 75 percent. Reductions of particulate nickel and chromium, however, were much less, with a removal of only 25-35 percent. The order for reduction of particulate metal species upon entering the West Pond is:

TABLE 2. COMPARISON OF SUMMARY STATISTICS FOR HIGHWAY RUNOFF AND MAITLAND RETENTION POND WATER

HEAVY METAL	METAL SPECIES	STORMWATER RUNOFF[1]		RETENTION POND WATER[2]		PERCENT REMOVAL IN POND	STATE OF FLA. CLASS III WATERS CRITERIA (2/1/83)
		MEAN (ug/1)	PERCENT OF TOTAL	MEAN (ug/1)	PERCENT OF TOTAL		
LEAD	Dissolved	33.0	18.2	15.0	67.6	54.5	
	Particulate	148	81.8	7.2	32.4	95.1	30 (Total)
ZINC	Dissolved	40.0	54.1	4.7	78.3	88.3	
	Particulate	33.9	45.9	1.3	21.7	96.2	30 (Total)
COPPER	Dissolved	28.6	74.1	14.4	86.2	49.7	
	Particulate	10.0	25.9	2.3	13.8	77.0	30 (Total)
NICKEL	Dissolved	2.5	73.5	1.6	72.7	36.0	
	Particulate	0.9	26.5	0.6	27.3	33.0	100 (Total)
CHROMIUM	Dissolved	2.5	59.5	2.2	62.9	12.0	
	Particulate	1.7	40.5	1.3	37.1	23.5	50 (Total)
IRON	Dissolved	77.9	20.6	18.4	29.2	76.4	
	Particulate	300	79.4	44.7	70.8	85.1	1000 (Total
ALUMINUM	Dissolved	125	22.3	58.0	37.1	53.6	
	Particulate	436	77.7	98.0	62.9	77.5	None
MANGANESE	Dissolved	2.7	28.3	4.5	26.3	-66.7[3]	
	Particulate	6.8	71.7	12.6	73.7	-85.3[3]	None
CADMIUM	Dissolved	1.7	77.3	0.73	89.0	57.1	
	Particulate	0.5	22.7	0.09	11.0	82.0	0.8 (Total)

NOTES:
1. Number of observations = 150
2. Number of observations = 30
3. Denotes an increase

Zn > Pb > Fe > Cd > Al > Cu > Ni > Cr >> Mn

Dissolved forms of zinc were removed to the greatest degree with an average removal of almost 90 percent. Dissolved iron was removed at an efficiency of 75 percent, followed by lead, copper, aluminum, and cadmium with removals of dissolved species ranging 50-60 percent. Removal efficiencies for dissolved nickel and chromium were very poor, with removals of only 36 and 12 percent respectively. The order for reduction of dissolved runoff species upon entering the West Pond is:

Zn > Fe > Cd > Pb = Al > Cu > Ni > Cr >> Mn

Studies by Yousef et al. (1985) as well as observations during this research suggest that the removal of dissolved metal species is rapid with as much as 90 percent removal occurring in four days. In the research by Yousef, et al., isolation chambers were placed in a newly constructed retention pond near Epcot Center. The isolation chambers were constructed of inverted polyethylene 200-liter barrels placed on the sediments - which isolated a 0.25 m^2 area of the sediment and the overlying water column. Chambers were constructed with both open and sealed bottoms to investigate the effects of sediments on heavy metal concentrations. The chambers were first installed then dosed with a solution of heavy metals. Periodic samples were collected and analyzed for metal content. A summary of their work is presented in Table 3.

Soluble concentrations of copper, zinc, iron, and lead were added to two of the test chambers in concentrations between 0.5 and 1 mg/l on 4/1/83. However, when the next sample was collected on 4/4/83, concentrations of copper, zinc, and lead had been substantially reduced by an average of 90 percent. By 4/18/83 (the next sample collection date) concentrations in the closed chamber were indistinguishable from the control which received no metal additions. No change was noted either with or without sediment contact in these metal concentrations throughout the test period, even when anaerobic conditions were established.

ACCUMULATION OF HEAVY METALS IN THE SEDIMENTS

HORIZONTAL DISTRIBUTION OF HEAVY METALS

Distributions of heavy metals in the top 1 cm of of the Maitland pond sediments suggest that upon entering the receiving water body, the majority of heavy metals associated with highway runoff settle out and are deposited near the point of input for the runoff. The distributions of selected heavy metals as a function of distance from the 45 cm RCP inlet are shown in Figure 2. This tendency was most obvious for lead and zinc which peaked in sediment concentrations at a distance of only 15 m from the inlet followed by a rapid decline in concentrations with increasing distance. Deposition patterns of the other metals measured were much less pronounced than those observed for lead and zinc. Chromium appeared to reach peak sediment concentrations at a distance of 30 m from the inlet with increases and decreases much less rapid than those observed for lead and zinc. Copper, nickel, and manganese did not

TABLE 3. UPTAKE AND RELEASE OF HEAVY METALS INSIDE ISOLATION CHAMBERS AT EPCOT POND

CHAMBER DESIGNATION	METAL	TOTAL METALS CONCENTRATION BY DATE IN 1983 (µg/1)										
		4-1	4-1* AFTER	4-4	4-18	4-21	4-25	5-5	5-9	5-12	5-19	5-24
Sediment Contact-Control (no metals added)	Cu	23	-	15	17	27	8	11	9	13	15	22
	Zn	7	-	9	5	6	4	4	3	4	4	4
	Fe	596	-	614	455	596	743	781	916	904	1118	1267
	Pb	23	-	32	26	23	27	24	28	26	23	23
Sediment Contact (metals added)	Cu	21	683	71	17	19	24	24	26	20	19	46
	Zn	14	857	82	10	10	11	10	5	12	5	10
	Fe	744	790	648	499	772	1059	1282	1703	2008	1654	1666
	Pb	24	904	93	23	29	41	27	39	37	29	22
No Sediment Contact (metals added)	Cu	23	590	61	19	17	22	14	64	28	28	41
	Zn	13	749	50	3	10	4	5	7	7	7	10
	Fe	401	468	720	617	788	612	360	341	535	269	300
	Pb	27	724	56	30	32	45	30	48	38	25	33
Pond	Cu	22	-	35	16	25	26	26	27	24	28	29
	Zn	12	-	4	0	6	4	3	2	3	4	3
	Fe	603	-	855	454	820	484	423	404	371	421	174
	Pb	28	-	52	21	36	48	46	49	51	44	45

<------Diffused Air-----> was Supplied <--------------Diffused Air was Shut-off------------->

* After addition of nutrient and heavy metal solution.

SOURCE: Yousef et al. (1985)

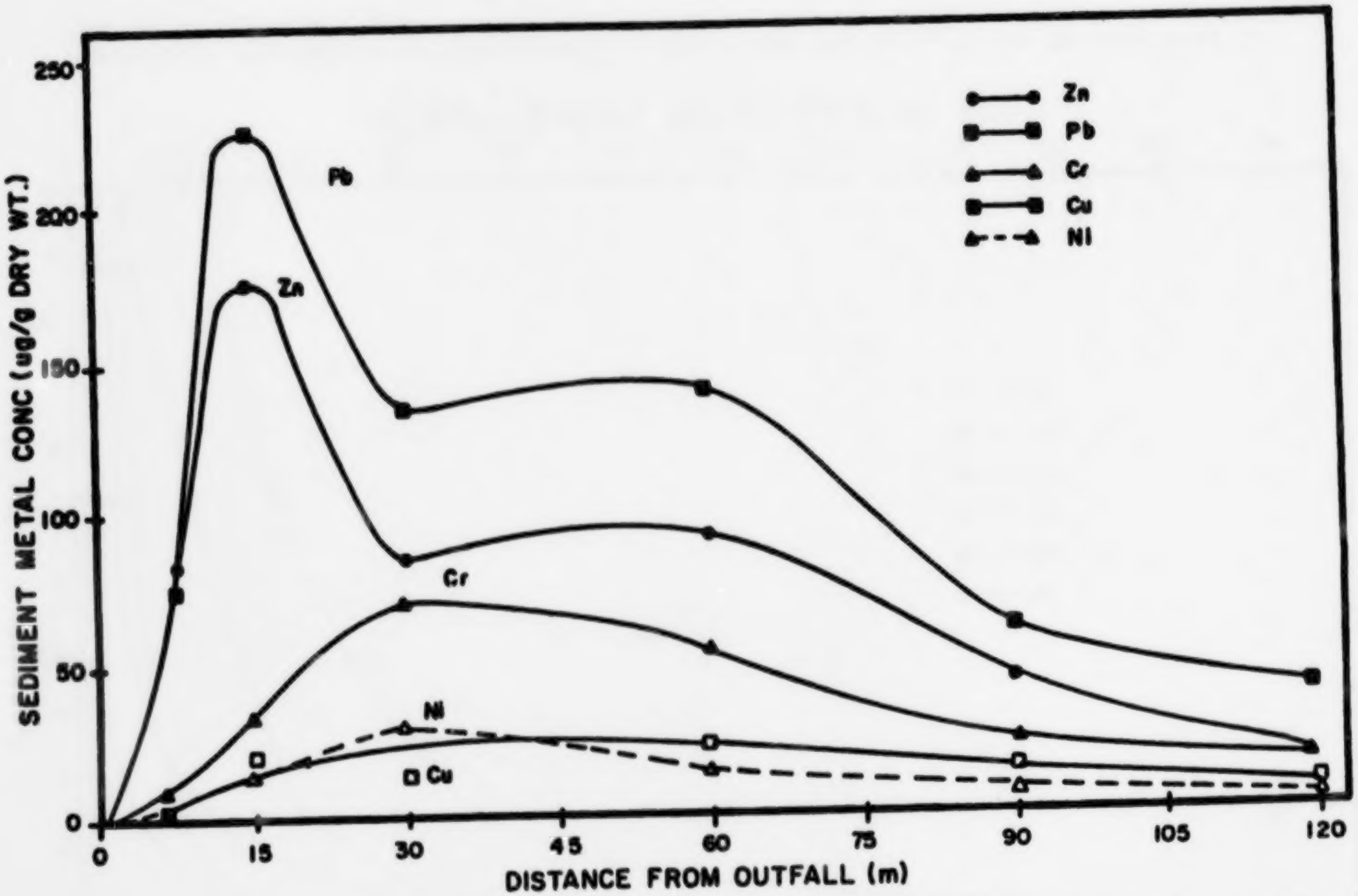

Figure 2. Sediment Concentrations of Selected Heavy Metals in the Top 1 cm of the Maitland West Pond as a Function of Distance from the Outfall.

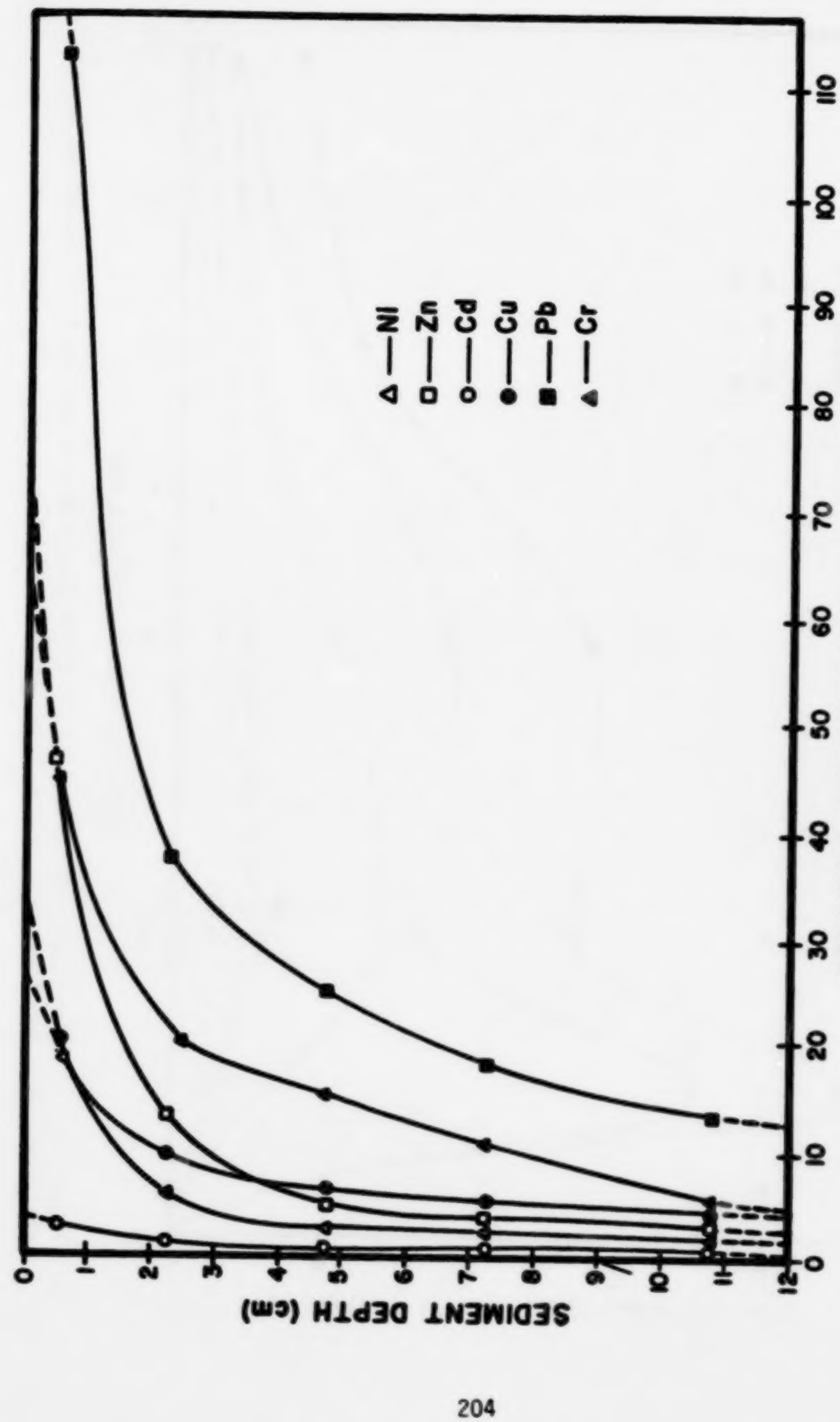

Figure 3. Attenuation of Heavy Metals in the Bottom Sediments of the Maitland Pond for Samples Collected 10/15/82.

TABLE 4. SUMMARY OF BACKGROUND AND RUNOFF RELATED METAL CONCENTRATIONS IN THE SEDIMENTS OF THE MAITLAND POND

SEDIMENT DEPTH	MEAN SEDIMENT METAL CONCENTRATION (µg/g DRY WT.)									NO. OF OBS.
	Cd	Zn	Cu	Al	Fe	Pb	Ni	Cr	Mn	
0 - 1 cm:										
Mean Conc.	2.20	45.4	19.1	49760	4554	112.7	16.5	44.7	43.9	138
Background	0.40	1.5	2.0	12058	654	7.9	1.98	7.63	1.65	6
Added Conc.	1.80	43.9	17.1	37702	3900	104.8	14.5	37.1	42.3	
1 - 3.5 cm:										
Mean Conc.	0.89	10.7	9.44	24574	1761	37.6	6.49	19.3	12.4	138
Background	0.40	1.5	2.00	12058	654	7.9	1.98	7.63	1.65	6
Added Conc.	0.49	9.2	7.44	12516	1107	29.7	4.51	11.7	10.8	
3.5 - 6 cm:										
Mean Conc.	0.56	6.49	7.50	18256	1303	24.5	4.15	15.4	6.86	138
Background	0.40	1.50	2.00	12058	654	7.9	1.98	7.63	1.65	6
Added Conc.	0.16	4.99	5.50	6198	649	16.6	2.17	7.77	5.21	
6 - 8.5 cm:										
Mean Conc.	0.45	4.64	5.07	17205	874	17.0	4.03	10.8	5.55	138
Background	0.40	1.50	2.00	12058	654	7.9	1.98	7.63	1.65	6
Added Conc.	0.05	3.14	3.07	5147	220	9.1	2.05	3.17	3.90	
8.5 - 13 cm:										
Mean Conc.	0.66	3.16	4.16	12485	482	13.5	3.31	4.01	4.96	138
Background	0.40	1.50	2.00	12058	654	7.9	1.98	7.63	1.65	6
Added Conc.	0.26	1.66	2.16	427	0	5.6	1.33	0	3.31	

appear to exhibit pronounced peaks in sediment concentrations, but seemed to settle out over a longer flow path length. However, in spite of the differences in behavior, most of the metals in the runoff water entering the Maitland pond were retained in the pond sediments within a distance of 60-90 m from the stormwater inlet.

Of the four metal species which exhibited the most rapid settling characteristics (lead, zinc, iron, and aluminum), all but zinc had particulate fractions in runoff which were near 80 percent of the total metal measured. The remaining metal species (nickel, chromium, and cadmium) which did not exhibit pronounced settling characteristics, were all present in highway runoff at the Maitland site predominately in a dissolved form with a small fraction of particulate species.

The results of the horizontal analyses of heavy metals suggest important design parameters for use in the design of retention basins to optimize removal of heavy metals. Designs should provide physical configurations where the flow velocity becomes very small to aid in sedimentation of particles. The distance from points of input to the discharge point from the pond should be maximized, and the design should minimize the possibility of short circuiting and avoid hydraulically dead zones.

VERTICAL DISTRIBUTIONS OF HEAVY METALS

The vertical distribution of heavy metals in the sediments of the Maitland West Pond was characterized by analysis of average sediment metal concentrations by layer on each of the three sample dates. The vertical distributions of selected heavy metals in the sediments of the Maitland pond are given in Figure 3. Aluminum was the most abundant metal present in the Maitland pond sediments at all depths, followed by iron. Lead was the third most abundant heavy metal present, followed by zinc and chromium, copper and nickel, and finally cadmium. Concentrations of cadmium were generally very small with many measured values, especially in the lower sediment depths, approaching the limits of detection. Measured concentrations of total heavy metals in the sediments of the Maitland pond exhibited highest concentrations in the surface layer with a rapid decline in concentration with increasing depth.

Background soil concentrations of heavy metals in the retention pond area were estimated from mean soil metal concentrations in core samples collected at depths of 3 m or greater during drilling of monitoring wells beneath the pond. These background concentrations were subtracted from the total sediment metal concentrations to provide an estimate of the added accumulations as a result of inputs of highway runoff. A summary of background and runoff related metal concentrations in the sediments of the Maitland pond is given in Table 4.

Sediment concentrations of runoff related heavy metals were also observed to decline rapidly with increasing depth. The rapid decline in concentrations was found to observe an exponential decay relationship with values of R-square in most cases in excess of 0.90 when fitted to the model: $\ln (C/C_o) = -K \times (\text{depth})$. A summary of the regression of statistics for the semi-log model is given in Table 5. Values of K, which are a measure of the rate of attenuation in sediment metal concentrations, indicated the following order of attenuation of total heavy metal content in the sediment layers:

Most Rapid   Least Rapid
Attenuation: Fe < Zn < Cd < Pb < Cr < Mn < Al < Ni < Cu: Attenuation

The calculated regression equations for runoff related metal accumulations were used to estimate the extent of metal migration from runoff related sources by estimation of the depths necessary to reduce runoff accumulations by 90 percent and 99 percent to values which are 10 percent and 1 percent above estimated background levels. All runoff related accumulations were reduced in concentration by 90 percent in the first 10 cm or less.

Although the substantial majority of metal species were attenuated in the first 5.0 cm of sediments, the depths necessary to achieve 99 percent reductions in runoff accumulations, based on the calculated regression equations, suggest that sediment concentrations of certain metals may be slowly migrating to lower depths. However, the vertical extent of this sediment-associated migration appears to be limited since all metal species were reduced in concentration by 99 percent within 20 cm or less. These calculations suggest a strong stability of the metal sediment associations since, after eight years of metal accumulations in the Maitland pond, most metals associated with sediments have remained in top 10 cm of the sediment layer.

CURRENT STABILITY OF METAL-SEDIMENT ASSOCIATIONS

The stability of metal-sediment associations was evaluated from the results of several different analyses. First, a sequential extraction procedure was used to determine metal speciation in composite samples of each of the five vertical core layers. Metal speciations were divided into soluble, exchangeable, carbonate bound, bound to Fe/Mn oxides, and organic bound fractions. It is generally believed that the stability of the metal-sediment associations increases in the following order: soluble < exchangeable < bound to carbonates < bound to iron and manganese oxides < bound to organic matter.

Fractional distributions of the total extracted heavy metals for each of the five extracted species are presented in Table 6. Most of the metal species tested, with the exceptions of lead, iron, and cadmium, appear to be predominantly associated with only one major fraction. For most metals, the dominant fraction is the one which is bound to Fe/Mn oxides. However, cadmium is predominantly associated with the exchangeable fraction. Lead also has a major association with this fraction. Alminum and iron appear to have significant fractions with organic particles. Very few of the heavy metals present in the sediments appear to be present in a soluble or carbonate form although cadmium, zinc and nickel had soluble fractions of approximately 10 percent. It appears certain that iron, manganese, and

TABLE 5. SUMMARY OF REGRESSION STATISTICS FOR ATTENUATION OF RUNOFF RELATED HEAVY METALS IN THE TOP 13 CM OF THE THE MAITLAND POND FOR A SEMI-LOG RELATIONSHIP FOR ALL THREE SAMPLE DATES COMBINED

HEAVY METAL	NO. OF OBS.	VALUE OF K FOR "BEST-FIT" EQUATION OF THE FORM:* $\ln (C/Co) = -KZ$	VALUE OF R-SQUARE
Cd	9	$\ln (Cd) = -0.374 (Z)$	0.821
Zn	11	$\ln (Zn) = -0.398 (Z)$	0.898
Cu	12	$\ln (Cu) = -0.286 (Z)$	0.877
Al	11	$\ln (Al) = -0.311 (Z)$	0.902
Fe	10	$\ln (Fe) = -0.549 (Z)$	0.821
Pb	12	$\ln (Pb) = -0.368 (Z)$	0.926
Ni	12	$\ln (Ni) = -0.304 (Z)$	0.898
Cr	9	$\ln (Cr) = -0.346 (Z)$	0.913
Mn	9	$\ln (Mn) = -0.327 (Z)$	0.895
Organic Content	12	$\ln (Org.) = -0.241 (Z)$	0.850

* Metal concentrations in units of μg/g; organic content in percent; and depth (Z) in units of cm.

TABLE 6. SPECIATION OF TOTAL HEAVY METAL CONCENTRATIONS IN THE SEDIMENTS OF THE MAITLAND WEST POND AS A FRACTION OF THE TOTAL METAL PRESENT

HEAVY METAL	PERCENT OF TOTAL EXTRACTED METAL CONCENTRATION					
	SOLUBLE	EXCHANGABLE	CARBONATE	BOUND TO Fe/Mn OX.	BOUND TO ORGANICS	TOTAL
Cadmium	15	52	12	10	11	100
Zinc	4	1	4	81	10	100
Copper	1	3	1	89	7	100
Aluminum	<1	<1	<1	74	26	100
Iron	<1	5	<1	52	43	100
Lead	1	44	1	52	2	100
Nickel	4	8	<1	82	6	100
Chromium	2	5	1	73	19	100
Manganese	1	9	1	86	3	100

organic content play dominant roles in regulating the mobility of metal species in the sediments of the Maitland pond.

In addition to the speciation analyses, an incubation apparatus was constructed which allowed simultaneous control of pH and redox potential in sediment suspensions to simulate metal adsorption or desorption under various environmental conditions. Sediment suspensions were incubated at pH values of 5.0, 6.5, and 7.5-8.0 which simulated current conditions of sediment pH in the pond, and at redox potentials of -250 mv, 0.0 mv, 250 mv and 500 mv, ranging from highly reduced to highly oxidized. A summary of metal release under these conditions for selected heavy metals is given in Figures 4 through 6. In general, release of most heavy metals was less than a few percent of the total sediment content under current conditions of pH (7.5-8.0). The influence of pH was found to be much more important than redox potential in regulating metal release for most metal species.

The results from the speciation and redox experiments combined with the analyses of the sediment metal concentrations presents evidence that under the current conditions of redox potential and pH within the sediments of the Maitland pond, metal species, with the exceptions of cadmium, lead, and manganese, are stable and exist in relatively immobile associations with Fe/Mn oxides and organic matter. Lead and cadmium are apparently held to a large degree in strong exchangeable associations. Changes in redox potential from strongly oxidized to strongly reduced conditions did not appear to affect the release of metals from the sediments under current pH values of 7.5-8.5. The release of most metals, except cadmium and manganese, from the sediment phase to the water phase was substantially less than 1% of the total metal present even after several weeks of incubation. However, cadmium and manganese appear to be less tightly bound to sediments than other metals. The release of both cadmium and manganese into solution from the sediment phase during incubation is equal to approximately 5% of the total metal present.

EFFECTS OF THE MAITLAND POND ON UNDERLYING GROUNDWATERS

A comparison of dissolved concentrations of heavy metals in the Maitland pond water with groundwater collected beneath the pond, represented by wells 2 and 3 combined, is given in Table 7. In general, concentrations of all heavy metals measured, except copper, were greater beneath the pond than within the pond. For certain heavy metals such as zinc, manganese, aluminum, and iron, measured concentrations in groundwaters were from 5 to 75 times as great as measured concentrations in the pond water.

Analysis of variance procedures were used to estimate the vertical extent of the migration of heavy metals in the aqueous phase. Zinc, manganese, aluminum, and iron were significantly higher in groundwater beneath the pond than in the pond at all depths tested. The extent of significantly higher concentrations of lead extended to the 0.5-1.0 m range. Copper was found to be significantly higher in the pond water than in groundwater in all analyses. Average concentrations of zinc, manganese, aluminum, and iron were found to be 4, 12, 8, and 50 times greater

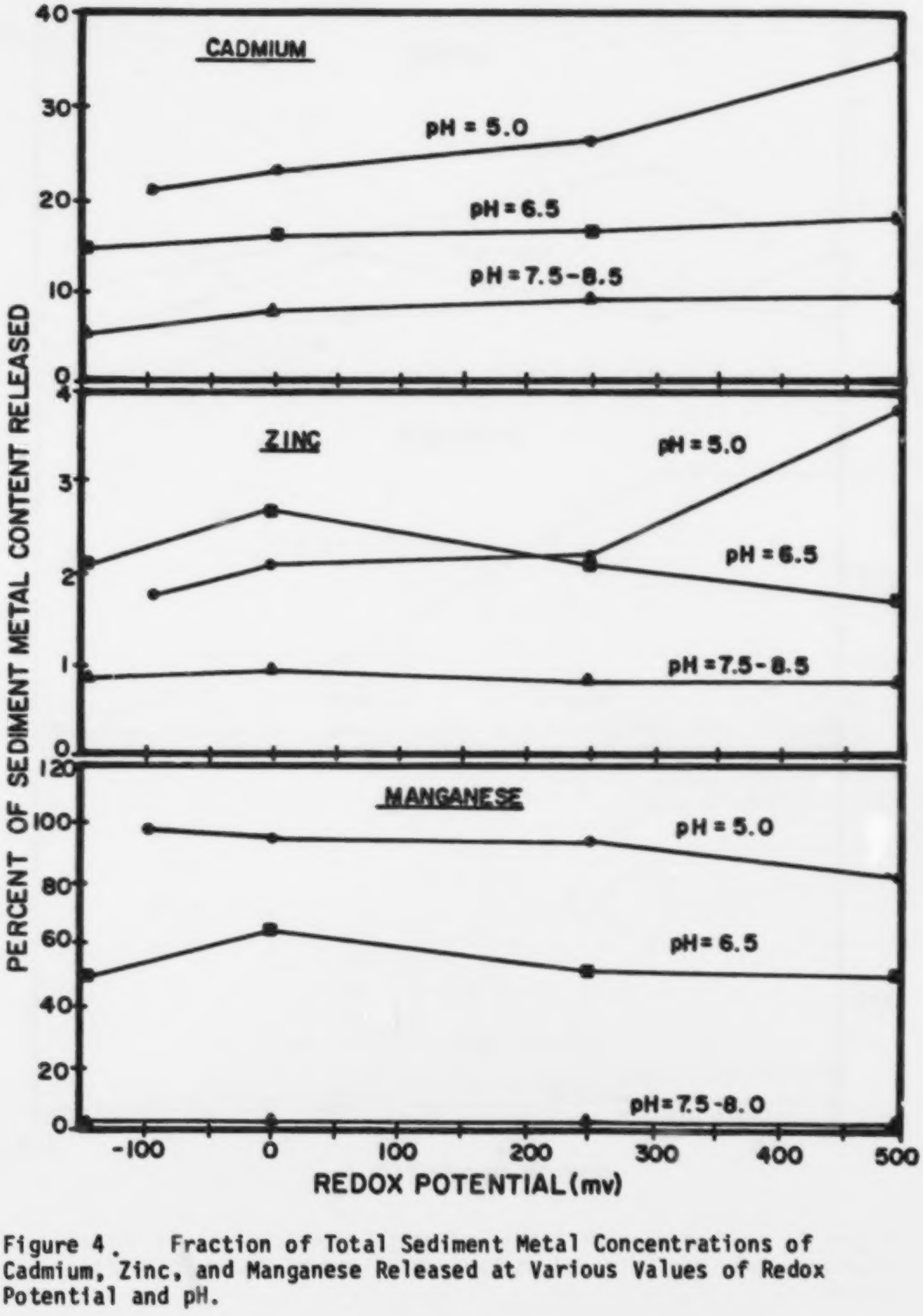

Figure 4. Fraction of Total Sediment Metal Concentrations of Cadmium, Zinc, and Manganese Released at Various Values of Redox Potential and pH.

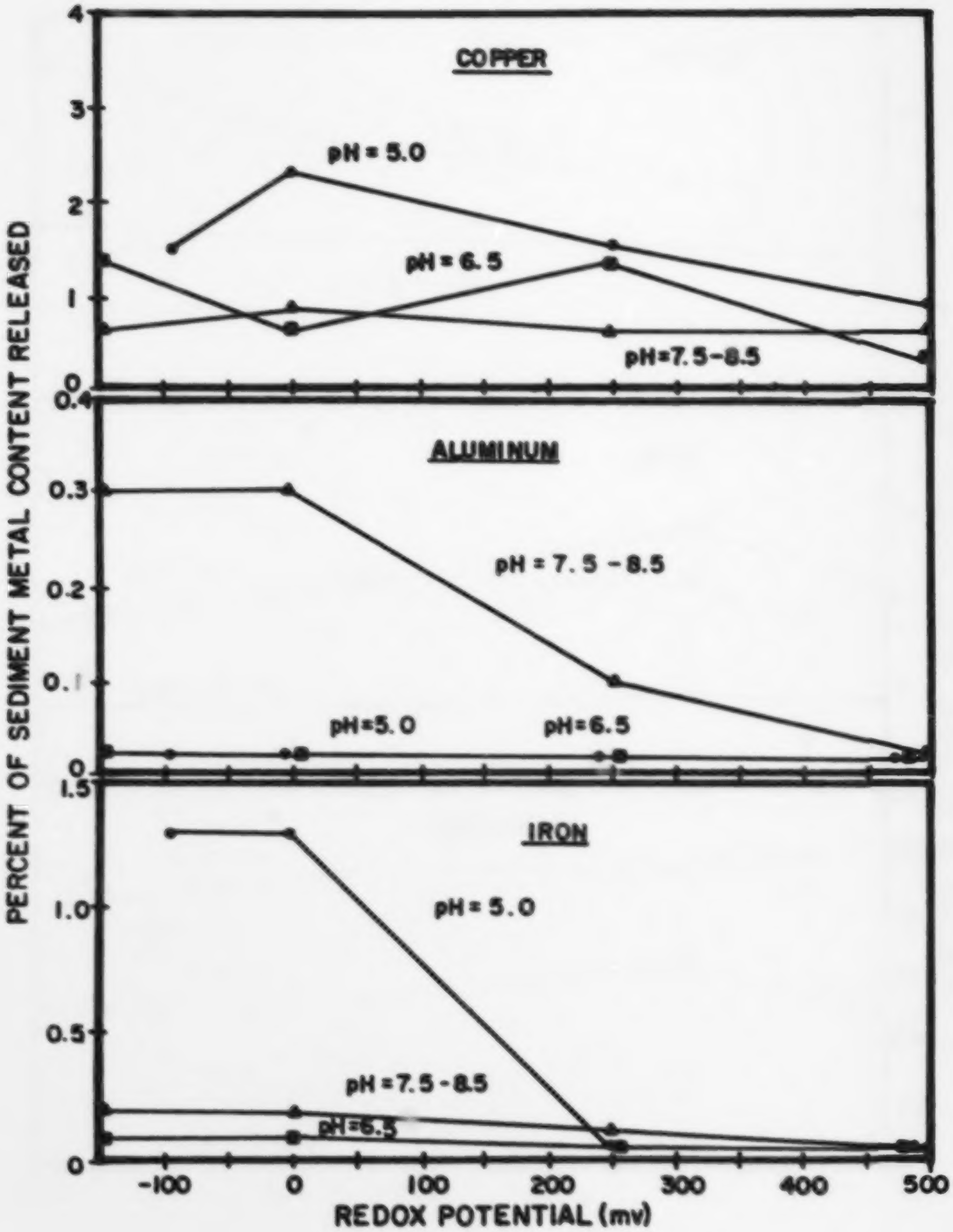

Figure 5. Fractions of Total Sediment Concentrations of Copper, Aluminum, and Iron Released at Various Values of Redox Potential and pH.

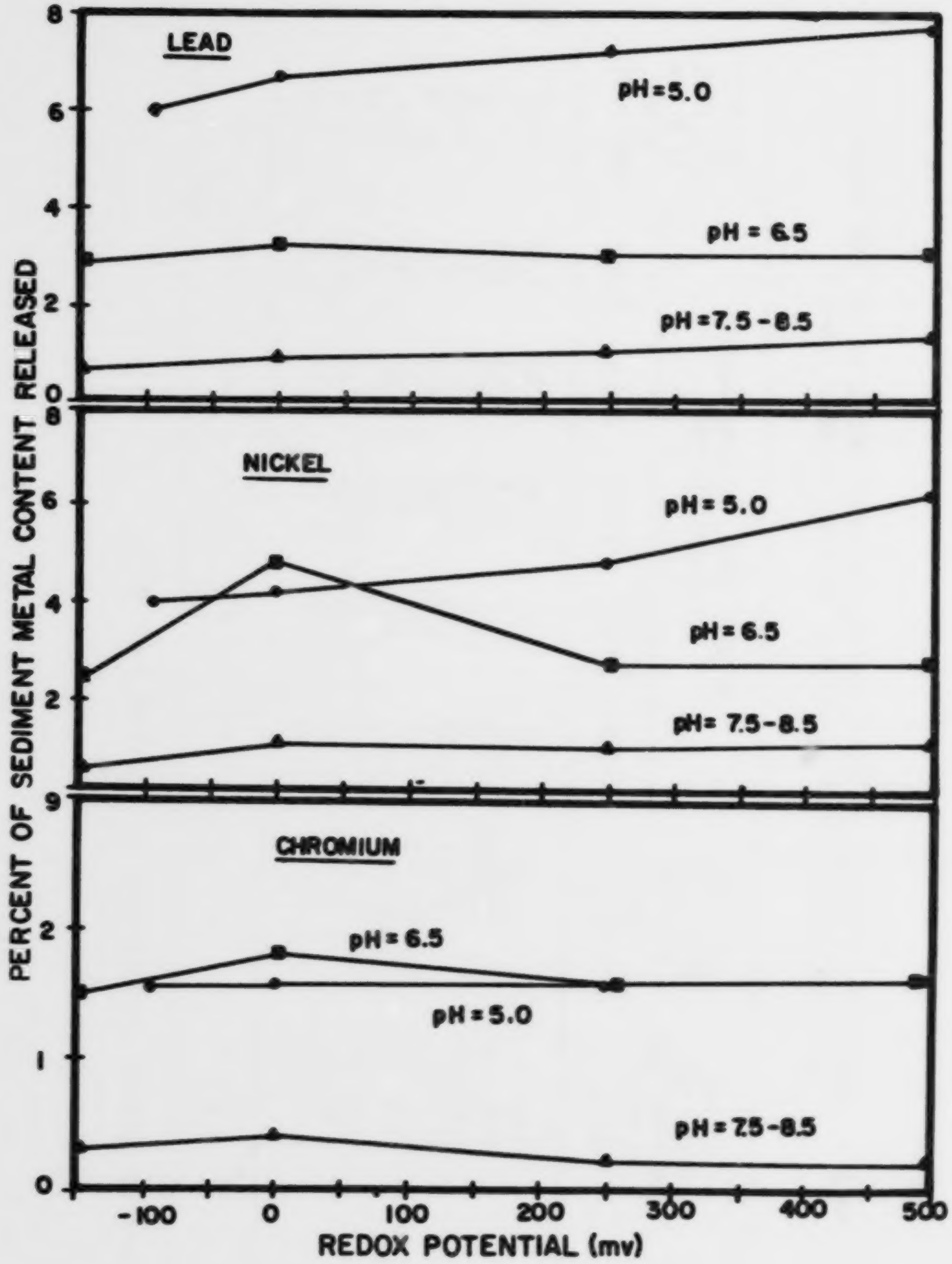

Figure 6. Fractions of Total Sediment Concentrations of Lead, Nickel, and Chromium Released at Various Values of Redox Potential and pH.

TABLE 7. COMPARISON OF DISSOLVED CONCENTRATIONS OF HEAVY METALS IN THE POND WATER WITH GROUNDWATER COLLECTED BENEATH THE POND IN WELLS 2 AND 3

HEAVY METAL	AVERAGE DISSOLVED CONCENTRATION IN POND (μg/l)	AVG. CONC. IN 0-1 cm SEDIMENT LAYER (μg/l)*	AVERAGE CONC. IN GROUNDWATER BENEATH THE POND					RATIO OF G.W. CONC. AT 0.1 m TO AVG. POND WATER
			0.1 m	0.5 m	1.0 m	3.0 m	6.0 m	
Cd	0.73	1,015	1.48	2.06	1.50	1.17	1.00	2.03
Zn	4.82	20,938	22.9	18.5	18.8	19.1	19.5	4.75
Mn	4.47	20,246	53.3	44.2	23.3	78.9	70.2	11.92
Cu	14.4	8,809	9.40	10.3	9.12	11.7	13.1	0.65
Al	57.9	22,949,100	709	742	192	89.2	543	12.25
Fe	18.4	2,100,285	1354	834	766	797	1912	73.6
Pb	15.0	51,977	24.4	24.8	20.5	11.2	10.9	1.63
Ni	1.62	7,610	2.88	2.63	1.98	1.94	2.13	1.78
Cr	2.18	20,615	3.02	4.15	2.60	1.29	1.54	1.39
pH	7.46		5.75	5.92	5.17	4.86	4.56	

* Average sediment concentration per liter of sediment.

respectively, in groundwater than in the pond water. Concentrations of cadmium and iron in groundwaters exceeded water quality criteria for Class III waters specified in Chapter 17-3 of the Florida Administrative Code.

Concentration ratios between the pond water and the groundwater indicate that all metal species, except copper, have a greater affinity for the groundwater phase than the pond phase and are leaching into groundwaters to some degree. The order of release potential of heavy metals into groundwater was estimated to be:

Least Mobile: Cu < Cr < Pb < Ni < Cd < Zn < Mn = Al < Fe: Most Mobile

and was found to be inversely related to the order of attenuation for metal species in the sediment phase. The magnitude of the release into groundwaters was found to closely correspond to the order of release predicted by the incubation experiments conducted under natural conditions of pH (7.5-8.0). However, in spite of the increased metal concentrations beneath the pond, the sediments are clearly the primary sink for heavy metals.

TRANSPORT OF HEAVY METALS IN GROUNDWATER FLOW

One of the objectives of this research was to monitor groundwater concentrations and flow patterns and to detect, if possible, the movement of heavy metals which leach into groundwaters. To aid in this detection, piezometers were installed at each well and a record of piezometric surface was made approximately on a bi-weekly basis during 1983. The average measurements are given in Table 8.

TABLE 8. AVERAGE MEASUREMENTS OF PIEZOMETRIC SURFACE AT MONITORING WELLS AT THE MAITLAND INTERCHANGE DURING 1983

Location	Piezometric Surface (m, MSL)
Well 1	27.35
Well 2	27.38
Well 3	27.37
Well 4	27.36
Well 5	26.87
Pond	27.38

As indicated in Table 8, little variation was measured in the piezometric surface between each well. As a result of this very small hydraulic gradient, horizontal movement of groundwater in the Maitland area was calculated to be less than 10 m per year. It appears that vertical up and down movement with changes in seasonal water table may be more important than horizontal movement. As a result, metal contamination of groundwaters appears to be very localized. Since the hydrologic conditions present at the Maitland site are similar to conditions at many other sites in Central Florida, it appears very unlikely that heavy metals from retention ponds along highway systems in the Central Florida area will pose a pollution hazard to nearby surface and groundwaters.

POTENTIAL FOR FUTURE MOBILIZATION OF HEAVY METALS

Natural aging processes within retention ponds as well as lakes result in the increased deposition of organic matter to the bottom sediments primarily as a result of the death and decay of both plant and animal matter. As these processes occur, it has often been observed that sediments become more reduced and decrease in pH. Although the incubation experiments indicated that most metal species are stable and tightly bound to sediments under current conditions of redox potential and pH, decreases in pH were found to increase the solubility of all heavy metals tested. Changes in redox potential produced no significant changes in release rates.

The results suggest that as the Maitland pond ages and accumulations of organic matter in the sediments begin to cause sediment pH values to decrease, mobilization of all metal species tested will increase and release to groundwaters may occur. Although all metals were found to increase in solubility with decreases in pH, the release was in general, only a small fraction of the total sediment metals present. For zinc, iron, aluminum, copper, and chromium, the maximum release was less than 3 percent of the total acid-extracted metal in the sediments, even at the most extreme pH value tested of 5.0. For nickel and lead, the release extended as high as 6-7 percent at a pH of 5.0. However, as seen in Figure 4, the release of cadmium and manganese into groundwaters can be expected to increase as the sediments become more acid. Manganese and cadmium were found to increase in solubility substantially as sediment pH decreases with almost total release of managanese and 25-35 percent release of cadmium at a pH of 5.0. Releases of this magnitude may produce measurable increases in groundwater concentrations beneath the pond. In the case of cadmium, a health hazard may be present under these extreme conditions.

The results suggest that maintenance procedures may be necessary after a period of time to remove the accumulated sediment deposits which may cause conditions of low pH and release of metals.

CONCLUSIONS

From the results obtained in these investigations the following specific conclusions were reached:

1. Measured concentrations of heavy metals in highway runoff collected at the Maitland Interchange during 1983-84 showed considerable variability between storm events as well as during storm events.

2. With the exceptions of copper and cadmium, the majority of metal species were present in a particulate form. The most common vehicle related heavy metals found in highway runoff at the Maitland site were lead, zinc, and copper which together accounted for approximately 91 percent of the dissolved heavy metals and 94 percent of the total metal concentrations.

3. The Maitland pond was found to be very effective in removal of heavy metal inputs from highway runoff. Particulate species of most metals were removed in the range of 75-95 percent with most of this mass retained in the pond sediments within a distance of 60-90 m from the stormwater inlet. In general, dissolved forms of heavy metals were removed to a lesser degree than particulate inputs with efficiencies near 50 percent for most metals.

4. Mean concentrations of heavy metals within the pond were within water quality criteria established in Chapter 17-3 of the Florida Administrative Code (F.A.C.) for Class III (recreational) waters.

5. Measured concentrations of total heavy metals in the sediments of the Maitland pond exhibited highest concentrations in the surface layer with a rapid decline in concentration with increasing depth.

6. After eight years of accumulations in the Maitland pond, most metals associated with sediments have remained in the top 10 cm of the sediment layer.

7. Under current conditions of redox potential and pH within the sediments of the Maitland pond, metal species, with the possible exceptions of cadmium and manganese, are stable and exist in relatively immobile associations with Fe/Mn oxides and organic matter.

8. In general, mean concentrations of all heavy metals measured, except copper, were greater in groundwaters beneath the pond than within the pond. Average concentrations of zinc, manganese, aluminum, and iron were found to be 4, 12, 8, and 50 times grater, respectively, in shallow groundwater than in the pond water. The extent of significantly higher groundwater concentrations of nickel, cadmium, and chromium extended to depths of 0.5-1.0 m, lead extended to the 1-3 m range, while zinc, aluminum, manganese, and iron were elevated in concentrations past the 6 m sample depth.

9. Violations of Class III water quality criteria were present for both cadmium and iron in groundwaters beneath the pond.

10. The horizontal movement of groundwaters in the study area was less than 10 m/year and as a result, the influence of the pond on groundwaters appears to be extremely localized.

11. As sediment accumulation occurs in retention ponds over time, the corresponding decreases in pH and ORP of the sediments will increase the

release of metal ions into groundwaters. The effect of reductions in pH were found to be more important than reductions in ORP in regulating the release of metal species.

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SIMPLE TROPHIC STATE MODELS AND THEIR USE IN WASTELOAD ALLOCATIONS IN FLORIDA

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ABSTRACT

Florida's Department of Environmental Regulation (FDER) uses a simple input-output trophic state model to establish wasteload allocation limits in cases involving wastewater discharge with a lake as the impacted waterbody. The model, SIMLAK, is based on in-lake chlorophyll a as predicted from loadings of nitrogen and phosphorus. A wasteload allocation performed by FDER for five potential dischargers to the Reedy Creek/Lake Russell system in Polk and Osceola Counties in central Florida resulted in phosphorus concentration limits much lower than levels achievable with available technology.

An independent analysis of the Reedy Creek/Lake Russell system was performed to examine the applicability of SIMLAK and other trophic state models, and to explore the potential impacts of various nutrient loading scenarios on Lake Russell. SIMLAK did not yield accurate predictions of chlorophyll a for Lake Russell based on present loadings, possibly because of the lake's highly colored water. The FDER allocation scenarios also required the nitrogen-to-phosphorus ratio in the lake to remain constant at the present ratio instead of controlling the limiting nutrient. Our analysis showed that variation of nitrogen-to-phosphorus ratios in the model could allow slight decreases in nitrogen concentration limits to offset increases in phosphorus concentrations (to achievable levels) with the same chlorophyll a prediction.

Alternative trophic state models based on one limiting nutrient did not yield better predictions of present conditions in Lake Russell. However, they did indicate that wastewater discharges at achievable phosphorus concentrations would not cause chlorophyll a in Lake Russell to exceed FDER's allowable limit.

The initial FDER allocation of 21.5 mgd would have required the five potential dischargers to meet a concentration limit of 2.15 mg/L for total nitrogen (TN), but total phosphorus (TP) limits ranged from 0.1 to 0.38 mg/L depending on distance from lake Russell. A final allocation agreed to by all parties allowed discharge of 22.5 mgd with concentration limits of 2.0 mg/L for TN and 0.5 mg/L for TP.

INTRODUCTION

REEDY CREEK/LAKE RUSSELL SYSTEM

The Reedy Creek drainage basin in central Florida extends from the southwest corner of Orange County, through Osceola County and into Polk County, where it joins the Kissimmee River system (Figure 1). Reedy Creek originates near the Disney World complex and flows southeastward through Reedy Creek Swamp into Lake Russell. Below Lake Russell the stream splits with approximately 30 % of the flow going to Cypress Lake and 70 % to Lake Hatchineha through the Dead River. Flow from these two lakes ultimately reaches Lake Kissimmee. The upper third of Reedy Creek has been channelized and modified by control structures, but the remainder of the creek flows through a natural stream channel.

Lake Russell is a highly colored lake with a surface area of about 300 ha, a mean depth of 1.83 m and a maximum depth of 2.6 m (Table 1). The annual hydraulic residence time is 30 days, and the range of retention times based on monthly flows is between 15 and 56 days. High levels of organic color limit Secchi depth to about 0.5 m and also appear to limit levels of chlorophyll a in the lake. Color ranged from about 300 to 500 PCU between 1979 and 1983, and chlorophyll a generally was less than 1 ug/L during the same period, except for one value of 16 ug/L in July, 1980 and one measurement of 13.5 ug/L in July, 1981.

WASTELOAD ALLOCATION PARTIES

In 1982 the City of Kissimmee was ordered by the State to eliminate its wastewater discharge to Lake Tohopekaliga; later in 1982 the city initiated discussions with the Florida Department of Environmental Regulation (FDER) concerning the possibility of discharge to Reedy Creek. At that time Reedy Creek Utilities was the only discharger to the creek, but several additional wastewater treatment plants also requested consideration from FDER for discharge capacity to Reedy Creek. The five utilities requesting consideration in the allocation included Reedy Creek Utilities, the City of Kissimmee, Central Florida Utilities, Poinciana Utilities, and Osceola Services Company. The FDER conducted a one-year water quality survey and

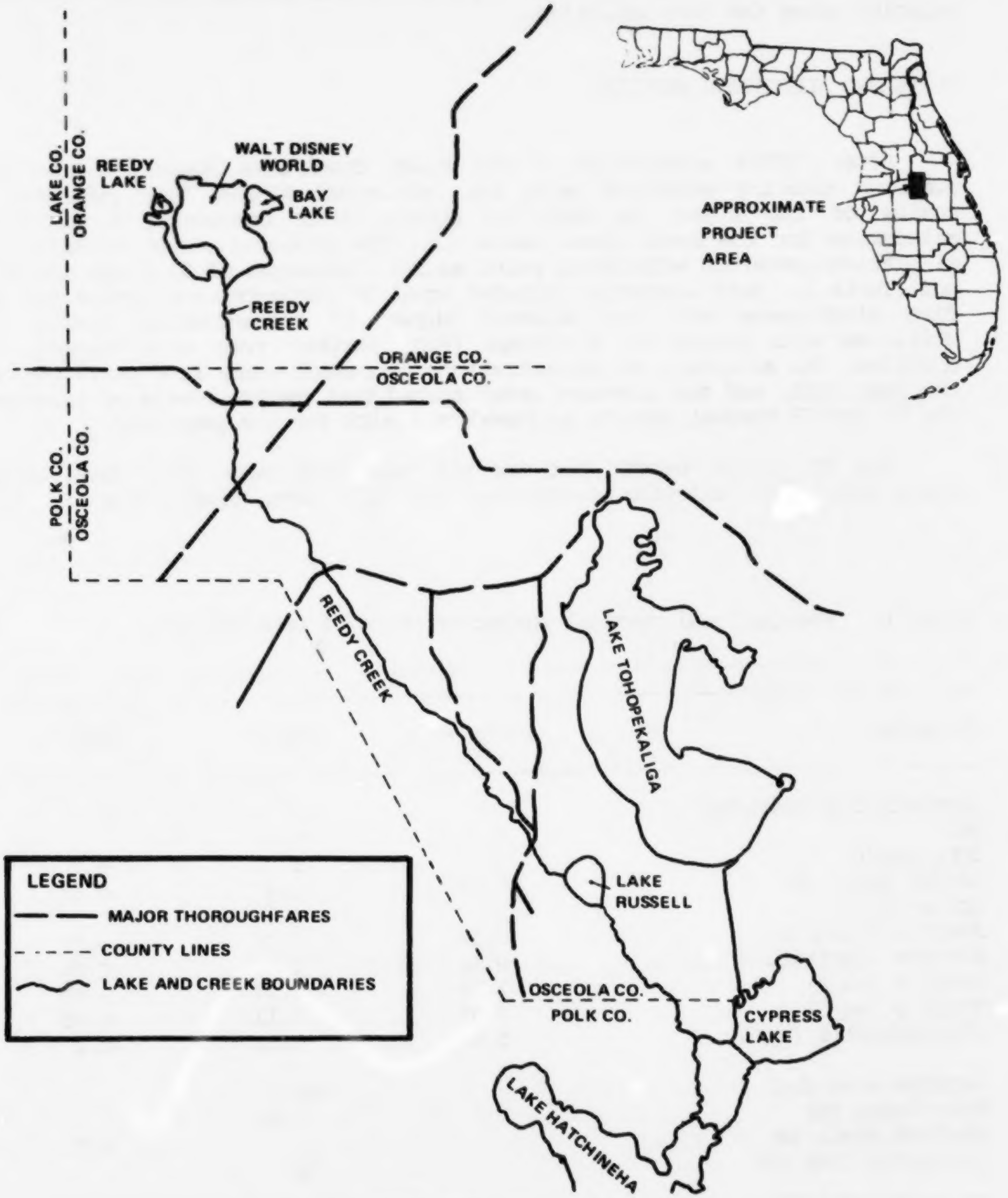

LAKE CO.
ORANGE CO.
REEDY LAKE
WALT DISNEY WORLD
BAY LAKE
REEDY CREEK
ORANGE CO.
OSCEOLA CO.
POLK CO.
OSCEOLA CO.
REEDY CREEK
LAKE TOHOPEKALIGA
LAKE RUSSELL
OSCEOLA CO.
POLK CO.
CYPRESS LAKE
LAKE HATCHINEHA
APPROXIMATE PROJECT AREA
LEGEND
MAJOR THOROUGHFARES
COUNTY LINES
LAKE AND CREEK BOUNDARIES

Figure 1. Reedy Creek/Lake Russell study area.

Russell, estimate the assimilative capacity of the system, and allocate that capacity among the five utilities.

WASTELOAD ALLOCATION RESULTS

After FDER's examination of the Reedy Creek/Lake Russell system and numerous modeling scenarios using the lake model SIMLAK, FDER published a notice of its intent to adopt an agency order concerning a wasteload allocation for the Reedy Creek basin (2). The proposed order included two allocation scenarios with total point source discharges of 21.5 mgd and 11.0 mgd (Table 2). Both scenarios included equal TN concentration limits for all five dischargers but they allowed higher TP concentration limits for utilities with points of discharge (POD) farther from Lake Russell. In addition, the allowable TP concentrations decreased every five years through the year 2005, and the proposed order stated that design levels of treatment for TN and TP removal were to be based on limits for the year 2005.

The TP limits established for the year 2005 were lower than levels achievable with existing technology for all five dischargers in both

Table 1. Physical and chemical characteristics of Lake Russell.

Parameter	STORET[1]	FDER[1]	BDA[2]
Conductivity (umho/cm)			95
pH	6.2	6.7	
BOD_5 (mg/L)	1.7	1.0	6.7
Secchi depth (m)	0.47	0.4	0.6
Color (PCU)	343	404	500
Ammonia N (mg/L)	0.15	0.36	0.07
Nitrate & Nitrite N (mg/L)	0.13	0.06	0.06
Total N (mg/L)	1.74	1.97	1.79
Total P (mg/L)	0.07	0.11	0.08
Chlorophyll a (ug/L)	5.5	0.02	0.2
Surface area (ha)		300	
Mean depth (m)		1.83	
Maximum depth (m)			2.7
Residence time (d)		30	

[1] Reference 1

[2] Field and laboratory results form Breedlove, Dennis & Associates, Inc. 1983 - 1984.

Table 2. Discharge limits of total phosphorus (mg/L) and total nitrogen (mg/L) in FDER notice of Intent to Adopt Agency Order (2).

Discharger	Flow (mgd)	TN	TP 1985	TP 1990	TP 1995	TP 2000	TP 2005
RCU	7.5	2.15	0.65	0.57	0.51	0.45	0.38
Kissimmee	6.0	2.15	0.17	0.15	0.13	0.12	0.10
CFU	3.0	2.15	0.29	0.25	0.23	0.20	0.17
Poinciana	3.0	2.15	0.17	0.15	0.13	0.12	0.10
OSC	2.0	2.15	0.41	0.36	0.32	0.28	0.24
	21.5						
RCU	4.0	2.54	0.62	0.44	0.32	0.20	0.08
Kissimmee	4.0	2.54	0.16	0.12	0.08	0.05	0.02
CFU	1.0	2.54	0.28	0.20	0.15	0.09	0.04
Poinciana	1.0	2.54	0.16	0.12	0.08	0.05	0.02
OSC	1.0	2.54	0.40	0.28	0.21	0.13	0.05
	11.0						

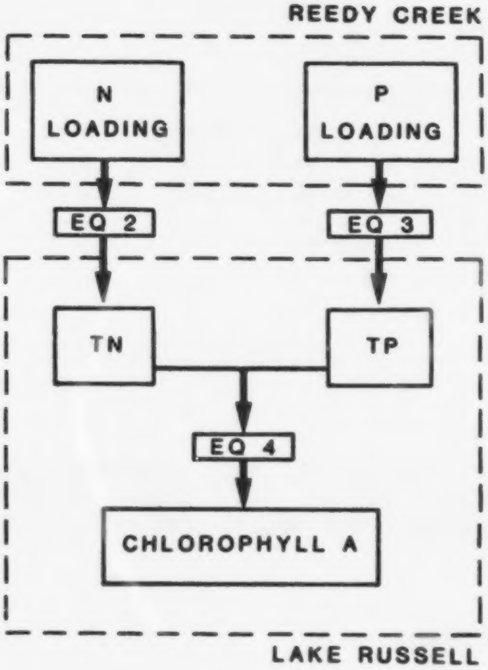
scenarios. Poinciana Utilities, Inc. subsequently filed a petition for an Administrative Hearing before the Florida Department of Administration to review FDER's proposed order and to request relief from the impacts of the proposed order (3).

INDEPENDENT EVALUATION

Poinciana Utilities, Inc. contracted with the authors of this paper to analyze the FDER wasteload allocation procedures as applied to Reedy Creek and Lake Russell, and to conduct an independent analysis of the potential impacts of wastewater discharge on Lake Russell and downstream lakes. This analysis included a thorough review of SIMLAK and the wasteload allocation process, modifications of SIMLAK, alternative trophic state models, and additional data related to potential impacts of wastewater discharge on fisheries, water clarity and nuisance aquatic plants.

SIMLAK

SIMLAK is a simple mathematical model that can be used to predict concentrations of TN, TP and chlorophyll a in a lake based on inputs of N and P, and physical characteristics of the lake such as hydraulic retention and basin morphometry (Figure 2). The SIMLAK equations were developed using


REEDY CREEK
N LOADING
P LOADING
EQ 2
EQ 3
TN
TP
EQ 4
CHLOROPHYLL A
LAKE RUSSELL

$$\text{EQ 1:}\quad R = 0.482 - 0.112 \ln \frac{\text{FLOW}}{\text{VOLUME}}$$

$$\text{EQ 2:}\quad TN = (1-R)\frac{\text{N LOADING}}{\text{FLOW}}$$

$$\text{EQ 3:}\quad TP = (1-R)\frac{\text{P LOADING}}{\text{FLOW}}$$

$$\text{EQ 4:}\quad CHLA = 35.95\left(\frac{TN}{3} + TP\right)\sqrt{\frac{\text{LENGTH}}{(\text{WIDTH})(\text{DEPTH})}}$$

Figure 2. Block diagram of SIMLAK model.

regression analysis of data from 33 Florida lakes included in the National Eutrophication Survey (4). Equation 1 is a nutrient retention factor that predicts the fractions of TN and TP retained in a lake as a function of its flushing rate (Figure 2). Equations 2 and 3 give predictions of in-lake TN and TP based on loadings, the retention factor, and the water flow rate into the lake. Equation 4 predicts chlorophyll a in the lake from the predicted values of TN and TP, and a shape term derived from the length, width and average depth of the lake.

WASTELOAD ALLOCATION PROCESS

The first step in the wasteload allocation process is to verify that SIMLAK works for the particular lake in question by using data on present inputs to predict existing concentrations of TN, TP and chlorophyll a in the lake. However, FDER has not established criteria for determining whether SIMLAK predictions are acceptable.

If the model produces "acceptable" results, the next step is to calculate allowable increases in nutrient loading to the lake. FDER does this by estimating non-point source (NPS) flows and inputs of nitrogen and phosphorus, and using SIMLAK to predict the amount of chlorophyll that should be produced by NPS inputs alone. A maximum increase in chlorophyll a of 2 ug/L is allowed if the predicted NPS chlorophyll a is less than 60 ug/L. In-lake concentrations of TN and TP are allowed to increase by the same percentage as the increase in chlorophyll a.

Equations 2 and 3 are re-arranged to yield nitrogen and phosphorus loads that would produce the TN and TP levels obtained in the previous step. New values of flow (Q) and retention (R) are used to reflect the combined total of NPS and design PS flows. When present NPS loads are subtracted from those total loads, the remainders are the allowable point source loads.

Of this allowable increase, only 75% is allocated among the PS dischargers, and 25% is set aside for increased NPS loads and as a safety factor. The allocatable load to the lake is divided among potential dischargers in the same proportion as their contributions to the total PS flow rate. The procedure described thus far produces an allocation of nitrogen, phosphorus and flow rates at the lake itself. FDER's next step is to extend the allocation to obtain limitations on discharge flows and nitrogen and phosphorus concentrations at the individual points of discharge to the system.

In the Lake Russell wasteload allocation, FDER concluded that nitrogen removal does not occur in Reedy Creek. Therefore the nitrogen concentration and flow limits calculated at the entrance to the lake also were applied without change to the points of discharge, and all potential dischargers received the same effluent TN concentration limits (Table 2).

However, FDER did conclude that phosphorus is removed during flow in Reedy Creek, and they calculated an average rate of phosphorus removal per mile of creek channel. This meant that higher phosphorus concentrations could be discharged farther upstream of Lake Russell and still theoretically meet the loading limits at the lake. The last column in Table 2 shows that using this method, potential dischargers with POD's far from Lake Russell would receive less stringent phosphorus concentration limits than dischargers whose assumed POD was closer to Lake Russell.

The final step FDER performed in the Reedy Creek wasteload allocation was an attempt to project NPS changes through the year 2005. They used information from the East Central Florida Regional Planning Council and from Osceola County to estimate the effect of future growth on NPS inputs of nitrogen and phosphorus to the Reedy Creek basin. FDER concluded that NPS loads of nitrogen will increase by 7%, and that NPS phosphorus loads will increase by 29% by the year 2005.

They then compared the projected NPS increases to the 25% reserve loadings set aside in the WLA process. In the case of nitrogen, future increases did not exceed the reserve amount. However, FDER's projected NPS phosphorus increases did exceed the 25% phosphorus reserve amount. When that occurred, FDER subtracted the additional NPS phosphorus increases from the amount originally allocated to point sources, thus reducing phosphorus limits as in the scenarios included in the FDER Notice of Intent.

INDEPENDENT EVALUATION

CRITICISMS OF SIMLAK

Our analysis of the Reedy Creek/Lake Russell WLA led to several conclusions regarding the model itself and its use for Lake Russell. Most nutrient loading models use different retention factors for TN and TP because of observations that they are retained to different extents in lakes where nutrient mass balances have been measured. Lakes generally retain more phosphorus than nitrogen, and thus it is inappropriate to use the same equation for both. In addition, the SIMLAK retention equation originally was developed for phosphorus retention in northern lakes, (5) and it does not give good predictions for Florida lakes.

SIMLAK also did not accurately predict chlorophyll a based on present nutrient inputs to Lake Russell. The model was designed to predict mean annual chlorophyll a, which has ranged from about 5 ug/L to <1 ug/L for Lake Russell. SIMLAK predicted 20.7 ug/L of chlorophyll a, but FDER considered that an acceptable fit because of the maximum values observed in 1980 (16 ug/L) and 1981 (13.5 ug/L).

The WLA precedure allows a maximum increase of 2 ug/L of chlorophyll a over the NPS chlorophyll a level in a lake, but SIMLAK is not capable of predicating chlorophyll a values with that degree of accuracy (4). The average error of SIMLAK chlorophyll predictions for the data set used to develop the model was ±21 ug/L. In addition to the problem of model accuracy, it would be very difficult to measure a 2 ug/L increase in chlorophyll. For example, mean chlorophyll a in Lake Kissimmee was 20.68 ug/L in 1979. The minimum statistically valid increase (at a 90% confidence level) with a monthly sampling program would be 4.0 ug/L. Even with a daily sampling schedule, 2.9 ug/L would be the minimum increase detectable at a 90% confidence level.

SIMLAK uses both nitrogen and phosphorus to predict chlorophyll a. Other nutrient loading models recognize that only one nutrient at a time limits the production of chlorophyll and they therefore have separate chlorophyll equations, one for phosphorus-limited lakes and another for nitrogen-limited lakes. However, because SIMLAK predicts chlorophyll as an additive function of TN and TP, the WLA procedure should allow for "tradeoffs" between the two nutrients such that an increase in one (above its "allowable" values) could be compensated for in SIMLAK by a decrease in the other (below its "allowable" value), the net effect of which would be to keep chlorophyll a levels predicted by SIMLAK within the allowable increase. Thus the WLA procedure does not allow the flexibility in computing nutrient loads that is inherent in the assumptions of the SIMLAK model.

According to the SIMLAK model, a small decrease in nitrogen loading could result in an increase in phosphorus loading to Lake Russell with no change in predicted chlorophyll a for the lake. The validity of the SIMLAK results should not be affected as long as N to P ratios remained within the range of N to P ratios found in the lake data used to develop the model equations. Nitrogen to phosphorus ratios vary widely in Florida lakes and they can change seasonally within a particular lake. There is thus no reason to require that N to P ratios in point source loadings should be the same as N to P ratios in NPS loads.

FDER evaluated discharge scenarios by comparing nutrient loadings to calculated loading limits rather than considering whether the chlorophyll a limit would be exceeded. When FDER projected the Lake Russell allocation to the year 2005, they added their predictions of future increases in NPS loads to present and proposed loads. After the projected increases in NPS phosphorus loads had equalled the 25% reserve phosphorus load, FDER subtracted subsequent increases from the amount allocated to dischargers in order to maintain the total phosphorus load within the WLA limit.

Instead of using that approach, we asked the question: If the PS phosphorus load were not decreased, and the total phosphorus load exceeded FDER's WLA limit, what would be the effect on Lake Russell's chlorophyll a? Using FDER's projections of future NPS loads and 24.5 mgd of PS discharge with TN = 2.15 mg/L and TP = 0.5 mg/L, we calculated that allowable chlorophyll a levels would not be exceeded through the year 2005 (Figure 3).

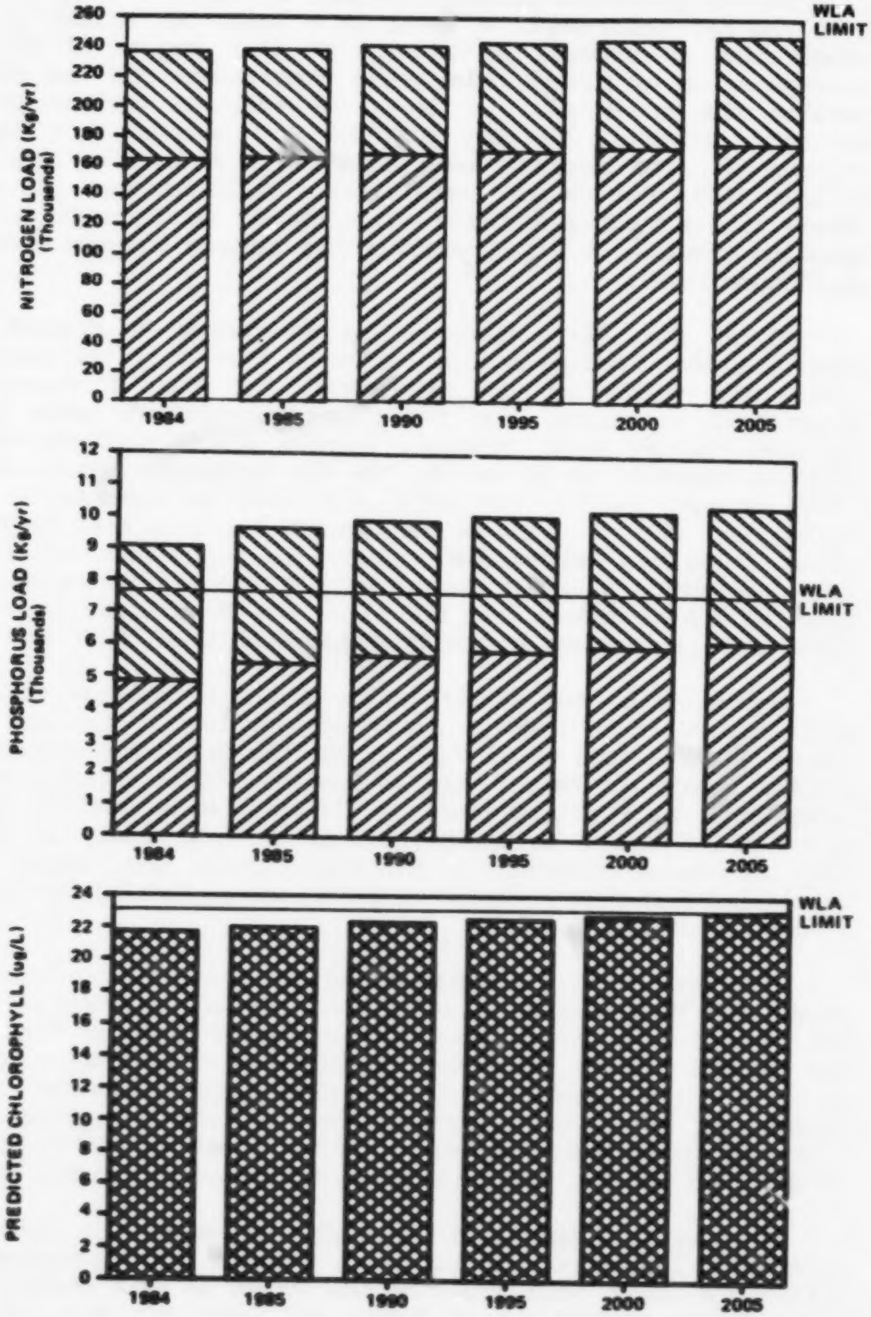


Figure 3. Predicted chlorophyll a in Lake Russell using SIMLAK equations with 24.5 mgd point source flow (TN = 2.15 mg/L, TP = 0.5 mg/L) and DER projections of future NPS loads.

The TP loading would exceed FDER's phosphorus limit throughout that time frame, but it would be compensated for by the fact that TN loads would remain less than the nitrogen limit.

In addition, FDER's estimate of future changes in NPS loading in the Reedy Creek basin did not include the effect of surface water management requirements on nutrient loadings. A similar analysis was performed (Harper, personal communication) using recent data on reduction of nutrient loadings in stormwater management systems similar to those that will be required in future developments. These results indicated that the land use changes projected by FDER would not cause increases in NPS loadings of nitrogen or phosphorus, and that both might actually decrease slightly. In such a case, predicted chlorophyll a in Lake Russell in the year 2005 would be at least 1 ug/L lower than FDER's wasteload allocation limit.

ALTERNATIVE MODELS

Numerous models are available for evaluating potential effects of PS discharges on lake trophic state and water quality. Some of the best-known models are accepted as reasonable empirical approaches to predicting the trophic state of temperate lakes (7,8) and several more recent models are modifications of their basic approach. The models have been adapted to Florida lakes by using a statistical approach to optimize equations that relate nutrient loading rates to lake response data (6). When results from those equations were compared to the SIMLAK equations we found that the modified Dillon and Vollenweider equations generally gave better predictions of nutrient and chlorophyll concentrations in Florida lakes than did the SIMLAK equations (Table 3).

The alternative equations also yielded over-predictions of chlorophyll a for Lake Russell based on present loading rates (Table 4), but they all predicted rather small increases in chlorophyll, N and P in the lake as a result of increased PS flows. We did not identify the reason that all the models predict higher chlorophyll levels than those observed in Lake Russell, but the high levels of color limit light penetration and may limit algal productivity. Additional possibilities include a limiting micronutrient such as molybdenum or zinc, the short water residence time, or high rates of zooplankton grazing. Nevertheless, the models all indicated that achievable concentrations of phosphorus in proposed discharges would have very small impacts on Lake Russell.

Additional regression models were used to evaluate the impact of increased PS loads on user-perceived values in Lake Russell. One equation relates Secchi depth in Florida lakes to chlorophyll and color (8):

$$\ln \text{Secchi depth} = 2.01 - 0.370 \ln \text{Chl } \underline{a} - 0.278 \ln \text{Color}$$

The model indicated that predicted increases in chlorophyll a in Lake Russell would result in decreases in water clarity too small to be observable to lake users.

Table 3. Predictive equations from SIMLAK and other Florida nutrient loading models.

Source	Equation	Coefficient of Determination
Nutrient Retention		
SIMLAK	$R = 0.482 - 0.112 \ln \frac{Q}{V}$	
	(for nitrogen)	$r^2 = 0.10$
	(for phosphorus)	$r^2 = 0.15$
Baker et al.	$R_N = -0.010 + 0.597 \log \frac{L_N}{Q}$	$r^2 = 0.51$
	$R_P = -0.056 + \frac{1.40}{1 + \rho W}$	$r^2 = 0.88$
Chlorophyll Prediction		
SIMLAK	$CHLA = 35.95 \left(\frac{TN}{3} + TP\right) \sqrt{\frac{L}{WZ}}$	$r^2 = 0.52$
Baker, et al.	$CHLA = 8.30 \left(\frac{L_N}{0.65\ Z + Q}\right)^{1.71}$	$r^2 = 0.69$
	$CHLA = 73.4 \left(\frac{L_P}{0.65\ Z + Q}\right)^{0.667}$	$r^2 = 0.60$

Table 4. **Summary of model predictions for Lake Russell with various discharge scenarios, (TP = 0.5 mg/L, TN = 2.15 mg/L).**

	Point Source Loadings (mgd)			
Predictions	**Present**	**18.5**	**21.5**	**24.5**
Total Nitrogen (mg/L)				
Observed	1.96	----	----	----
SIMLAK	1.95	2.02	2.02	2.02
Baker et al.	1.64	1.63	1.63	1.63
Total Phosphorus (mg/L)				
Observed	0.110	----	----	----
SIMLAK	0.068	0.070	0.074	0.077
Baker et al.	0.073	0.078	0.078	0.079
Chlorophyll a (ug/L)				
Observed	3.0 (16.0 Max)	----	----	----
SIMLAK	20.7	21.5	21.6	21.7
Baker et al. N	34.8	35.3	35.3	35.2
Baker et al. P	13.7	14.2	14.2	14.6
Nutrient Retention Factor				
SIMLAK	0.1996	0.1709	0.1658	0.1610
Baker et al. Rn	0.2496	0.2408	0.2395	0.2383
Baker et al. Rp	0.2532	0.2234	0.2184	0.2137

Regression models also were constructed to relate nutrient concentrations to lake coverage by nuisance aquatic weeds from data contained in the FLADAB lake data base developed at the University of Florida (9). No significant relation was found between nutrient concentrations and areal coverage or percent coverage by water hyacinths or hydrilla. The lack of a nutrient - aquatic weed relationship and the small changes predicted in Lake Russell nutrient levels suggested that proposed PS loadings were unlikely to affect the status of aquatic weeds in Lake Russell.

Finally, we used equations relating fish biomass and percent biomass of sport fish, forage fish and rough fish to the chlorophyll trophic state index in Florida lakes of varying trophic state (10). We found that sport fish biomass expressed as a percent of total biomass decreases as trophic state index increases (Figure 4). However, the predicted change is caused by an increase in rough fish biomass, and the actual biomass of sport fish (kg/ha) appears to be insensitive to changes in trophic state index.

SUMMARY AND CONCLUSIONS

The wasteload allocation procedure applied by FDER resulted in overly restrictive discharge limits to Reedy Creek because FDER did not take advantage of the ability of SIMLAK to vary N to P ratios and because of the 25% reserves of N and P. Alternate models and the SIMLAK equations indicated that point source flows of 24.5 mgd with nutrient concentration limits of 0.5 mg/L for TP and 2.15 mg/L for TN should cause only slight increases in nutrient concentrations in Lake Russell that would not be noticeable to users of the lake.

The Administrative Hearing was settled by a consent agreement that allowed 22.5 mgd of PS flows to Reedy Creek with TP of 0.5 mg/L and 2.0 mg/L of TN. The wasteload allocation procedures have been modified recently by FDER to allow for more outside involvement, but the SIMLAK model has not been changed.

Based on the Reedy Creek/Lake Russell wasteload allocation, we recommend that FDER develop several models better suited to different types of Florida lakes (N limited, P limited, highly colored lakes). Their approach should allow reasonable variations in lake conditions such as N to P ratios in order to attain allocations with achievable nutrient concentration limits. FDER also should develop procedures for model calibration and criteria for judging the acceptability of model predictions. Their models should be capable of predicting changes in lake conditions that are equivalent to the increase allowed by the WLA process.

The work described in this paper was not funded by the U.S. Environmental Protection Agency and therefore the contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

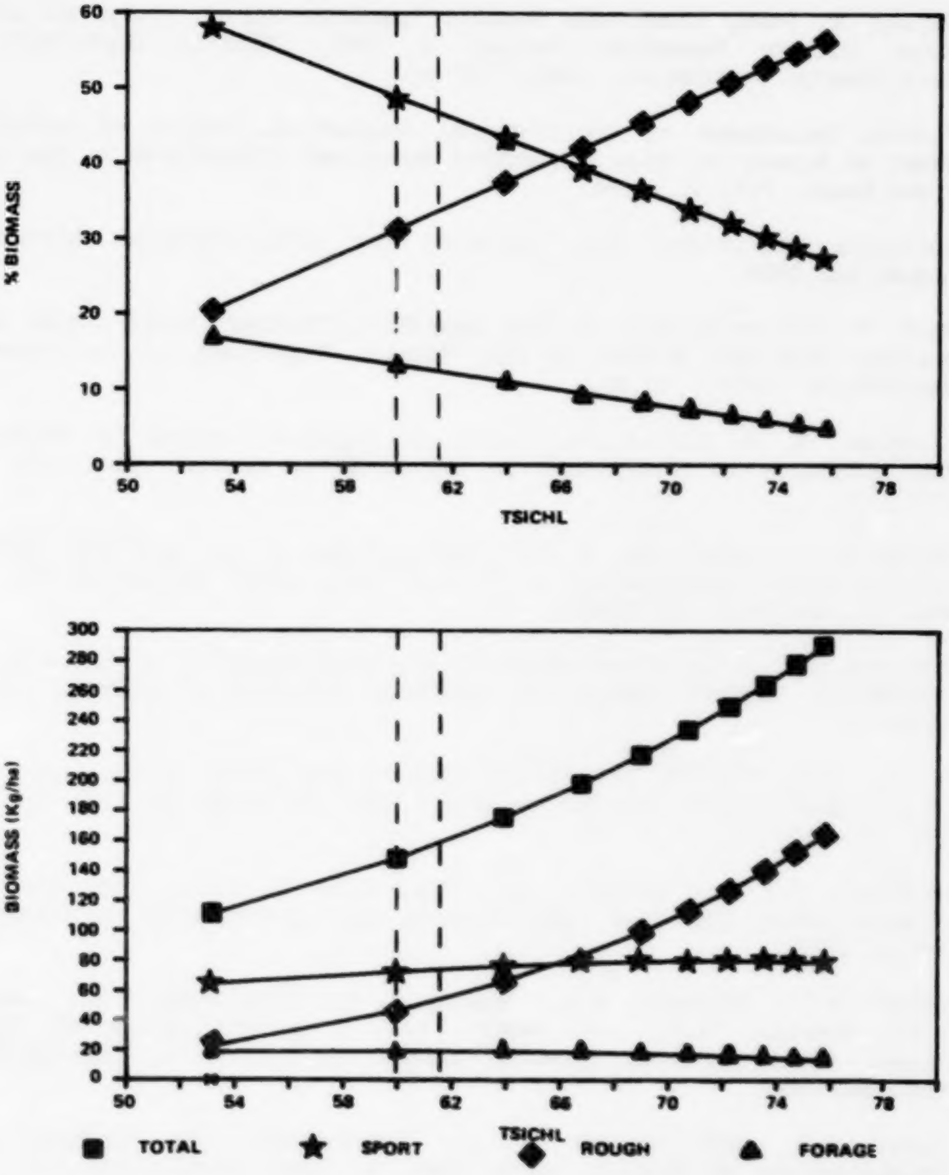


Figure 4. Fish-Trophic state relationship in Florida lakes (from Bays and Crisman 1983; Dashed lines represent approximate range of TSI for Lake Russell based on SIMLAK predictions of chlorophyll).

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MODEL COMPLEXITY FOR TROPHIC STATE SIMULATION IN RESERVOIRS

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ABSTRACT

Frequently model users are faced with choosing an appropriate model structure and level of complexity for a particular application. Increased model complexity generally implies incorporation of more portions or a larger portion of the real world, but also generally implies an increased number of rate coefficients and input parameters, each of which has some associated uncertainty. This paper examines this issue through mathematical simulation of trophic state in reservoirs with two models of different levels of complexity. Analysis and comparison of the models' output is accomplished through various statistical measures and techniques including root mean squared error, regression analysis with student's t test for slope and intercept, and difference of means. This study revealed that for predicting steady-state total phosphorus concentration, no advantage to using the more complex model over the simpler model was observed. For time-variable simulation, it was determined that the models were substantially in agreement for long term averaging (i.e. greater than 10 years), but that such models do not agree for short term simulation nor even for the frequency distribution of phosphorus concentrations within that simulation period. In less biologically productive oligotrophic systems, the level of model complexity appeared to be of lesser importance than for more biologically productive eutrophic systems. The nonlinearity of the phosphorus uptake relationship is demonstrated to be more pronounced in eutrophic than in oligotrophic systems, and explains the discrepancy between the models' predictions.

INTRODUCTION

Frequently model users are faced with choosing an appropriate model for their particular application. For trophic state determination in impoundments, the choices range from the steady state total phosphorus loading diagrams (1, 2) to the time-variable ecosystem models (3, 4, 5, 6, 7, 8). In many cases a user may assume that the more complex the model, the more accurate or reliable the simulation. This may not always be true as even the most complex models are never mathematical clones of real physical systems. Thomann (9) aptly states, "A mathematical model is simply an analytical abstraction of the real world. As such, it does not pretend to incorporate all phenomena but rather abstracts only those portions of the real world that are relevant to the problem under consideration."

Although increased complexity generally implies incorporation of more portions or a larger portion of the real world, it must be realized that increased complexity also generally implies an increased number of rate coefficients and input parameters to utilize the model. Since the numerical value of each of these has some associated uncertainty, then it is entirely possible that uncertainty in the model output may also increase with complexity. The parabolic curve of Figure 1 represents a feasible picture of the relationship between model complexity and model uncertainty. Of course, as the state of our knowledge of individual biological, chemical, and physical processes improves throughout time, so too will our ability to mathematically describe these processes and hence to build more certain complex models.

This paper does not attempt to resolve the entire dilemma, but rather to examine it through mathematical simulation of trophic state in reservoirs by comparing the output of two models of different complexity. The comparisons are made under steady-state and time-variable conditions. For the latter, two types of analyses are conducted: (1) Direct comparison of the models' predicted concentrations at corresponding times, and (2) Comparison of the frequency distribution of the models' predicted concentrations over the entire simulation period. The methodology and conclusions are applicable to modeling water quality constituents in general.

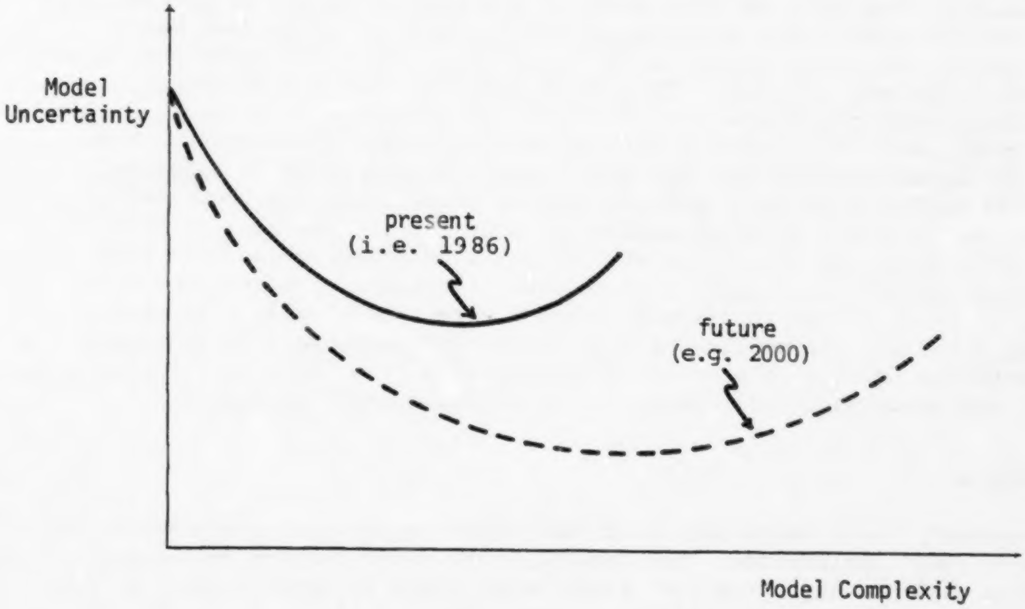


Figure 1. A Relationship Between Model Complexity and Uncertainty

METHODOLOGY

Two contrasting models of phosphorus dynamics in reservoirs are considered. Both models account for time-variable reservoir volume, influent and effluent flow rate, and influent phosphorus concentrations. The first model (10) represents a reservoir as a single fully mixed system and models a single water quality variable, total phosphorus (TP). The model is depicted in Figure 2. The change in TP with time is modeled as the sum of the influent load, minus the effluent load minus a net decay of phosphorus (e.g. due to sedimentation), according to the following equation:

$$\frac{d(V \cdot TP)}{dt} = Q_i \cdot TP_i - Q_e \cdot TP - \frac{v_s}{\bar{H}} \cdot TP \cdot V \qquad (1)$$

where V = reservoir volume
t = time
Q_i = influent flow rate
Q_e = effluent flow rate
TP_i = influent phosphorus concentration
TP = phosphorus concentration in the reservoir and in the effluent
v_s = apparent settling velocity
$\bar{H}$ = mean depth of reservoir

Typical values for v_s reported in the literature are 10 to 16 m/yr (1, 11, 12). The form of the model is similar to those previously presented by Vollenweider (1) and Chapra (13). It will be referred to hereafter as the Total Phosphorus Model, TPM.

The second model (14) represents a reservoir as having two fully mixed layers, an epilimnion and a hypolimnion, during periods of thermal stratification. When temperature gradients are small or absent, for example during overturn, the reservoir is represented as a single fully mixed layer. Three forms of phosphorus - organic phosphorus (OP), dissolved inorganic phosphorus (DIP), and particulate inorganic phosphorus (PIP) - and dissolved oxygen are modeled in each layer. The model is depicted in Figure 3. The model equa-

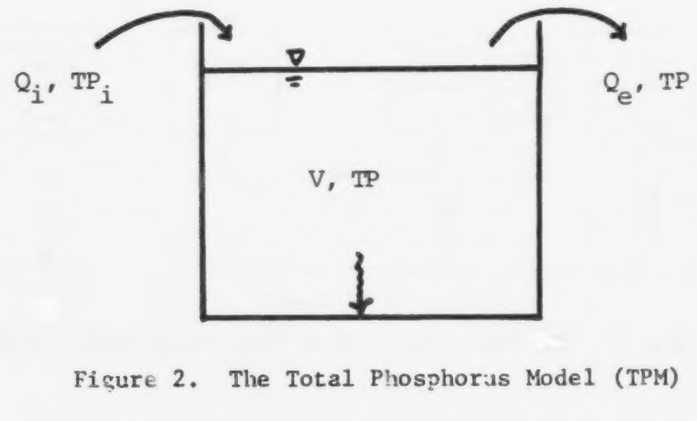
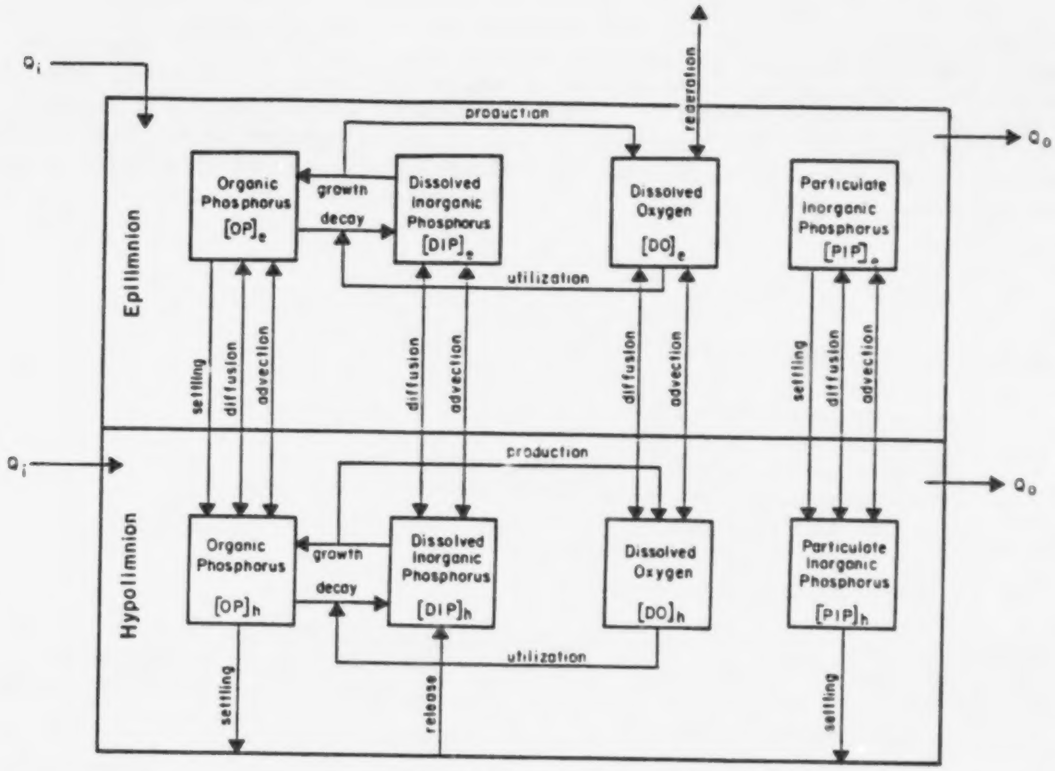
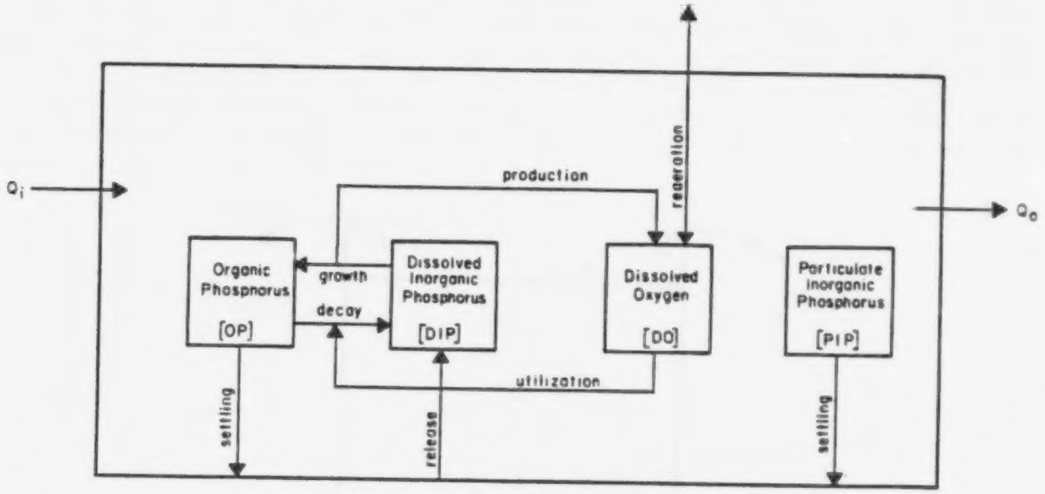


Figure 2. The Total Phosphorus Model (TPM)



Q_i
reaeration
production
Q_o
Epilimnion
Organic Phosphorus $[OP]_e$
growth
decay
Dissolved Inorganic Phosphorus $[DIP]_e$
Dissolved Oxygen $[DO]_e$
Particulate Inorganic Phosphorus $[PIP]_e$
utilization
settling
diffusion
advection
diffusion
advection
diffusion
advection
settling
diffusion
advection
Q_i
production
Q_o
Hypolimnion
Organic Phosphorus $[OP]_h$
growth
decay
Dissolved Inorganic Phosphorus $[DIP]_h$
Dissolved Oxygen $[DO]_h$
Particulate Inorganic Phosphorus $[PIP]_h$
utilization
settling
release
settling

Stratified Model



reaeration
production
Q_i
Q_o
Organic Phosphorus [OP]
growth
decay
Dissolved Inorganic Phosphorus [DIP]
Dissolved Oxygen [DO]
Particulate Inorganic Phosphorus [PIP]
utilization
settling
release
settling

Fully Mixed Model

Figure 3. The Multi-Component Phosphorus Model (MCPM)

tions have been presented previously in Griffin and Ferrara (14). The model will hereafter be referred to as the Multi-Component Phosphorus Model (MCPM).

TPM may be considered very nearly the simplest of model structures, i.e. a low level of complexity. MCPM provides a higher level of complexity incorporating characteristics of nutrient cycling and biological transformations. However, it is not as complex as some of the ecosystem models cited above. MCPM was developed to simulate seasonal changes in water quality, yet still be simple and economical enough for long term simulation on the order of decades. TPM was also derived for long term simulation, yet because of its simple structure, would not be expected to consistently predict seasonal changes in water quality accurately.

TPM requires specification of one rate coefficient, i.e. v_s. The quotient $\frac{v_s}{H}$ is sometimes reported as a constant, σ, a first-order net sedimentation rate coefficient. At steady-state, the solution to TPM is

$$TP = \frac{TP_i}{1 + \frac{v_s}{H}\frac{V}{Q}} = \frac{TP_i}{1 + \sigma\bar{t}} \qquad (2)$$

where $\bar{t}$ is the detention time, $\frac{V}{Q}$. Vollenweider (15) has reported, based on data from many lakes, that

$$\sigma = \frac{1}{\sqrt{\bar{t}}} \qquad (3)$$

Hence, knowledge of the detention time of a particular waterbody leads to an estimate of the single reaction rate coefficient necessary to utilize the total phosphorus model.

STATISTICAL MEASURES

Various statistical techniques exist for comparing data sets whether the data are actual observations or the results of model simulation. Statistical measures used in this paper include the following (16, 17).

1. Confidence Interval about Mean

For a limited number of data points, X, one can identify an interval within which it can be said, with a certain amount of confidence, that the true mean of X, μ_x, will reside. This interval is given as follows:

$$\bar{X} - t_{\gamma/2}\frac{S}{\sqrt{N}} < \mu_x < \bar{X} + t_{1-\gamma/2}\frac{S}{\sqrt{N}} \qquad (4)$$

where $\bar{X}$ = the sample mean
S = the sample standard deviation
t = the percentile of the student t distribution with N-1 degrees of freedom and γ equal to 100 minus the % confidence level.

When comparing two data sets, if their intervals overlap, then the sample means are not significantly different, and vice versa.

2. Root Mean Squared Error

A convenient method of comparing the difference between two data sets is to compute the root mean squared error, R, where

$$R = \sqrt{\frac{\sum_{i}^{N} (X_{1i} - X_{2i})^2}{N}} \tag{5}$$

The subscripts 1 and 2 refer to data sets 1 and 2 respectively. The significance of R is pictured in Figure 4 where it is shown to represent the deviation from perfect agreement between two data sets. Since R has the same units as X (e.g., concentration), it is useful for comparison to express a relative error, RE, where

$$RE = \frac{R}{\bar{X}} \tag{6}$$

Note that either $\bar{X}_1$ or $\bar{X}_2$ could be used in the denominator of equation (6).

3. Regression Analysis

A further test of agreement can be made by regressing the data as presented in Figure 4 and comparing the intercept, a, and slope, b, of the regressed line with the theoretical values of α and β, respectively. A two-tailed t test with N-2 degrees of freedom and the following statistics is conducted,

$$t_b = \frac{b - \beta}{S_b} \quad \text{and} \quad t_a = \frac{a - \alpha}{S_a} \tag{7}$$

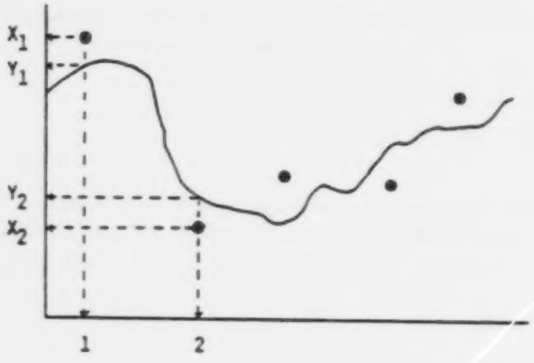
where S_b and S_a are the standard deviation of the slope and intercept, respectively. For $t < t_{\gamma/2}$, there is no significant difference between the calculated and theoretical value for the particular parameter. As a note of caution, it is entirely possible to obtain values of b and a not significantly different than 1.0 and 0, respectively, with poor correlation between the two data sets (i.e., $r^2 < < 1.0$, where r^2 is the square of the correlation coefficient).

4. Difference of Means

Computing the difference between the means of two data sets, $d = \bar{X}_1 - \bar{X}_2$, leads to the following statistic

$$t_d = \frac{d - \delta}{S_d} \tag{8}$$

ROOT MEAN SQUARED ERROR


X_1
Y_1
Y_2
X_2
1
2

$$R = \sqrt{\frac{(Y_1 - X_1)^2 + (Y_2 - X_2)^2 + \ldots (Y_N - X_N)^2}{N}}$$

REGRESSION ANALYSIS

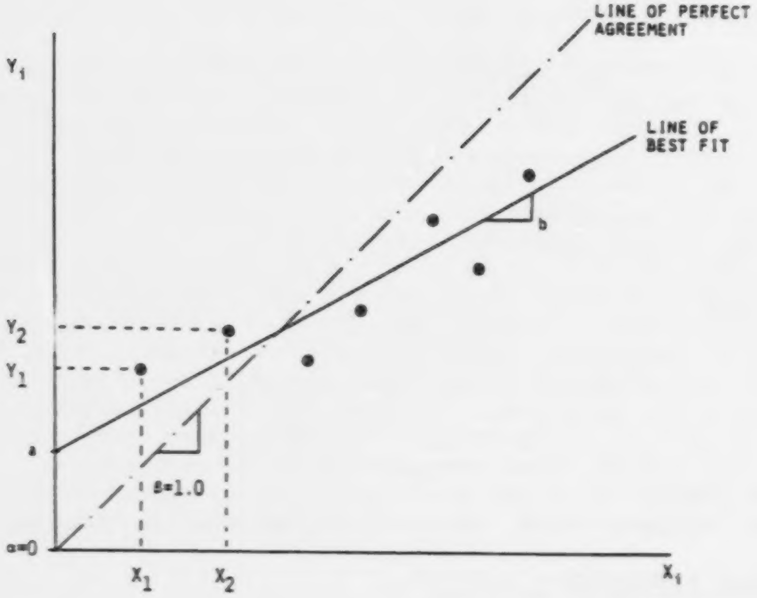

LINE OF PERFECT AGREEMENT
LINE OF BEST FIT
Y_i
Y_2
Y_1
a
b
$\beta=1.0$
$\alpha=0$
X_1
X_2
X_i

Figure 4. Root Mean Squared Error and Regression Analysis

where δ = true difference between data set means
S_d = the standard deviation of d which is given by a pooled estimate of the population variance.

If the variance of the two data sets are assumed equal, then

$$t_d = \frac{d - \delta}{\sqrt{(N_1-1)S_1^2 + (N_2-1)S_2^2}} \sqrt{\frac{N_1N_2\ (N_1+N_2-2)}{N_1 + N_2}} \qquad (9)$$

Note that for perfect agreement between data set means, δ=0. Then a value of $t_d < t_{\gamma/2}$ with (N_1+N_2-2) degrees of freedom suggests that there is no significant difference between data set means, and vice versa.

STEADY STATE SIMULATION

The first test of the models was aimed at comparing response under steady-state conditions. The MCPM was run to steady state utilizing an impoundment and its calibrated rate coefficient values identified in Griffin and Ferrara (14). The predicted effluent steady state total phosphorus concentration thus obtained, i.e. TP_{MCPM}, was compared with the analogous value calculated from equations (2) and (3), i.e. TP_{TPM}. This was conducted for a total of eighteen (18) tests listed in Table 1. These tests represent six reservoir volumes, each of which was subjected to three flow rates, providing a wide range of reservoir volumes, flows, and hence detention times.

As noted in Table 1, the models have similar means and standard deviations. Tests for the 95% confidence interval about the mean and the difference between means concludes that there is no significant difference. The TPM output however results in a higher coefficient of variation (CV) indicating greater variability in its values. The range of steady state concentrations for TPM was between 21.9 and 48.3, whereas for MCPM the range was 29.0 to 43.0. Linear regression between the two data sets results in a slope and intercept of 1.12 and -5.48 respectively. The t test discussed above concludes that these values are not significantly different than 1.0 and 0.0, respectively. Finally, the root mean squared error between the two models is 4.8, leading to a relative error of only 13%. Hence it may be concluded that the two models do not predict significantly different results under steady-state conditions. Thus for simulating steady-state total phosphorus concentration, no advantage to using the more complex multi-component model over the simpler total phosphorus model is demonstrated. Of course, if the user is interested in the distribution of phosphorus among its various organic and inorganic forms, the more complex model is required.

TIME-VARIABLE SIMULATION - CASE I

Problems involving trophic state analysis of reservoirs often require time-variable analysis. Mathematically, both TPM and MCPM are capable of simulating short term variations. Practically and ecologically, it is obvious that their capabilities are different. In its development TPM was intended for assessment in long term simulation and not necessarily to pre-

TABLE 1. STEADY STATE SIMULATION RESULTS

Test	$\overline{t}$(yr)	σ(eq.3)	TP_{TPM}(ug/l)	TP_{MCPM}(ug/l)
I	0.06	4.13	48.3	43.0
II	0.10	3.12	45.4	39.1
III	0.12	2.92	44.7	41.1
IV	0.20	2.26	41.6	41.1
V	0.21	2.21	41.3	37.6
VI	0.27	1.91	39.4	41.2
VII	0.34	1.71	37.9	37.7
VIII	0.35	1.68	37.7	41.6
IX	0.41	1.56	36.6	30.1
X	0.43	1.52	36.2	42.1
XI	0.48	1.44	35.5	37.9
XII	0.62	1.27	33.6	38.5
XIII	0.75	1.15	32.1	39.0
XIV	0.82	1.10	31.5	29.0
XV	1.37	0.85	27.6	29.0
XVI	1.92	0.72	25.2	29.3
XVII	2.47	0.64	23.3	29.9
XVIII	3.01	0.58	21.9	30.5
Mean			35.5	36.5
Standard Deviation			7.59	5.26
Coefficient of Variation			0.21	0.14

dict water quality changes from one year to the next, but rather to predict the frequency distribution of water quality (as measured by total phosphorus concentration) due to some reservoir management scenario. MCPM was intended for simulation of seasonal changes in water quality, yet be economical enough for long term simulation as well.

It might be expected from the steady-state analysis presented previously in this paper and from the objectives of the two models that their predictions for long term average concentrations should be in agreement. In such cases it would be expedient to use TPM. However, it might further be expected that as the period of simulation decreases, the model predictions would begin to diverge. At that stage, the model user must choose between TPM and MCPM, and would likely accept the latter. The critical question then becomes how to identify this point of divergence and how different the model predictions really are. This may be accomplished through comparison of the model predictions for simulations of varying time periods.

MCPM was run for a fifty-seven year simulation utilizing the rate coefficients previously defined (Figure 5). From the model output a mean total phosphorus concentration was calculated, $\overline{TP}_{MCPM} = 27.2$ ug/l. TPM was then run for the same conditions. However, it was first necessary to choose a value for the sedimentation rate coefficient. Equation (3) is inappropriate for two reasons, (1) it was derived for steady-state analysis, and (2) the concept of detention time becomes meaningless under time-variable simulation unless of course a temporally averaged flow rate and reservoir volume are used to compute it. Values reported for v_s in the literature were similarly determined from steady-state data. Chapra (13) utilized a

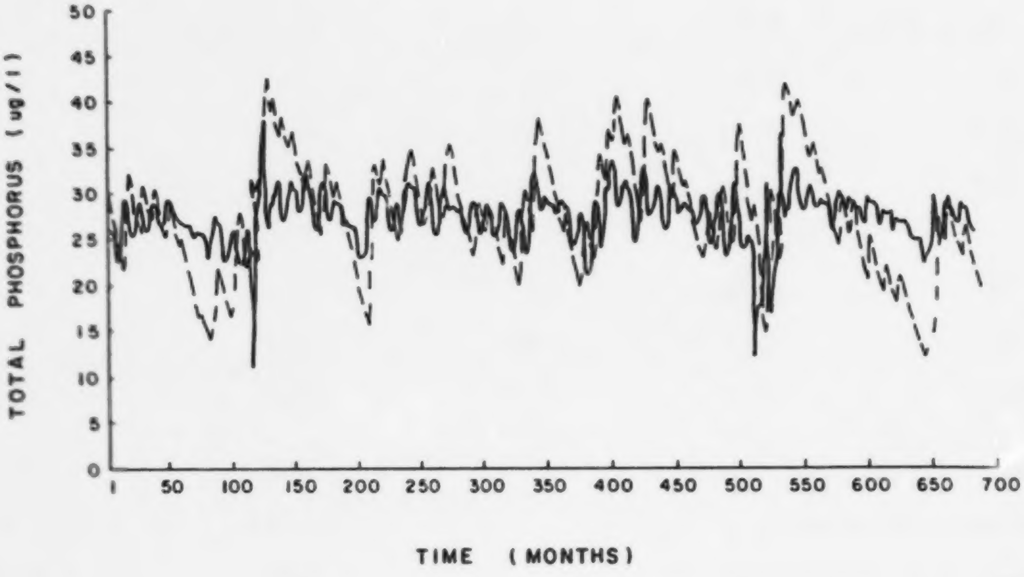


Figure 5. Time Variable Simulation - Case 1

(dashed line represents TPM; solid line represents MCPM) value of v_s = 16 m/yr in a time-variable total phosphorus model of the Great Lakes. Since there is no à priori method for choosing a value for v_s and a sufficiently large data base for model calibration did not exist, it was decided for this study to adapt a value which results in $\overline{TP}_{TPM} = \overline{TP}_{MCPM}$ for the 57 year simulation. This of course forces the two models to agree in the long term average but does not prejudice their relative agreement or disagreement for shorter periods or for the frequency distribution of concentrations over the long term. A value of v_s = 9 m/yr results in $\overline{TP}_{TPM}$ approximately equal to $\overline{TP}_{MCPM}$ (Table 2). This value is indeed close to the reported steady-state range of 10 to 16 m/yr and the time variable value utilized by Chapra (13).

The 57 year simulation may be broken down into various averaging periods for comparison of the two models. For any averaging period of simulation years there will be (57-x+1) values of $\overline{TP}_{TPM}$ and $\overline{TP}_{MCPM}$ for comparison. For example, for a 57 year averaging period there is one value each for $\overline{TP}_{TPM}$ and $\overline{TP}_{MCPM}$, i.e., 27.5 and 27.2 ug/l respectively. For a 25 year averaging period, there will be 57-25+1 = 33 values for each model. For monthly averaging periods of course there will be 57x12 = 684 total phosphorus concentrations for each model.

The root mean squared error for various averaging periods is provided in Table 3. Note that the relative error or disagreement between the models increases up to 19% when monthly average TP concentrations are compared, whereas the disagreement is only 5% for the 25 year average concentrations. The results of the regression analysis are also presented in Table 3. Slopes and intercepts are significantly different than one and zero for the monthly and 1 year groups but not for the 4, 10 and 25 year groups. This statistical analysis verifies the hypothesis that the degree of agreement between the two models does decrease significantly as the time period of simulation for comparing average total phosphorus concentrations is also decreased.

An analysis of the frequency distribution of predicted monthly average total phosphorus concentrations for each model was also conducted. Although it is shown above that the monthly average predictions in individual months do not agree, it may be that the distributions of those predictions will agree. This may be satisfactory for certain management decisions where for

TABLE 2. $\overline{TP}_{TPM}$ AS A FUNCTION OF v_s - CASE I

v_s(m/yr)	$\overline{TP}_{TPM}$(ug/l)
5	34.5
7	30.6
8	29.0
9	27.5
10	26.2
12	23.9
15	21.1

TABLE 3. STATISTICAL ANALYSIS OF TIME VARIABLE SIMULATION - CASE I

Averaging Group	R	RE	Slope	Intercept
Monthly	5.05	0.19	1.36	-9.32
1 year	4.66	0.17	1.74	-19.86
4 year	3.84	0.14	1.75*	-20.11*
10 year	2.19	0.08	1.23*	-5.18*
25 year	1.48	0.05	1.54*	-13.51*
57 year	0.30	0.01	-	-

*Indicates value not significantly different than 1.0 for slope or 0.0 for intercept at 95% confidence level.

example it is only necessary to know the percent of time concentration exceeds a certain standard. The data were grouped into intervals of 0.5 ug/l, (Figures 6.a and 6.b). Note that the data are approximately normally distributed but that the characteristics of the two models are interestingly different. As in the steady-state simulations, although both models have essentially the same mean value, TPM is characterized by a larger standard deviation and hence coefficient of variation. Consequently, the distribution of predicted monthly average total phosphorus concentrations appears to be different. This was tested by conducting a regression analysis comparing the frequency of occurrence in respective concentration intervals. The results of this analysis reveal a slope and intercept of 0.32 and 7.43 respectively, values which were further determined to be indeed statistically significantly different than the theoretical values of 1.0 and 0.0 respectively. Therefore, in addition to the fact that the two models are predicting significantly different concentrations in corresponding individual months, they are also predicting significantly different distributions of concentrations over the 57 year simulation period.

TIME VARIABLE SIMULATION - CASE II

The above time-variable simulation was conducted on an impoundment which might be considered eutrophic, i.e. total phosphorus concentrations generally greater than 20 ug/l. In order to compare the models in a less nutrient-rich system, the same simulations were conducted with the influent phosphorus concentration reduced to one-fourth of its previous value. The model simulations are presented in Figure 7. A value of $\overline{TP}_{MCPM}$ = 10.07 ug/l (very nearly oligotrophic conditions) was obtained. Interestingly, the calibrated value for v_s was found to be 3 m/yr (Table 4). Because of the reduced influent phosphorus load, this system is far less biologically active. Since the role of biological uptake and deposition has diminished, the system must be characterized by a smaller net sedimentation/decay rate coefficient. In the previous system, a value of v_s = 9.0 m/yr was obtained and was in close agree-

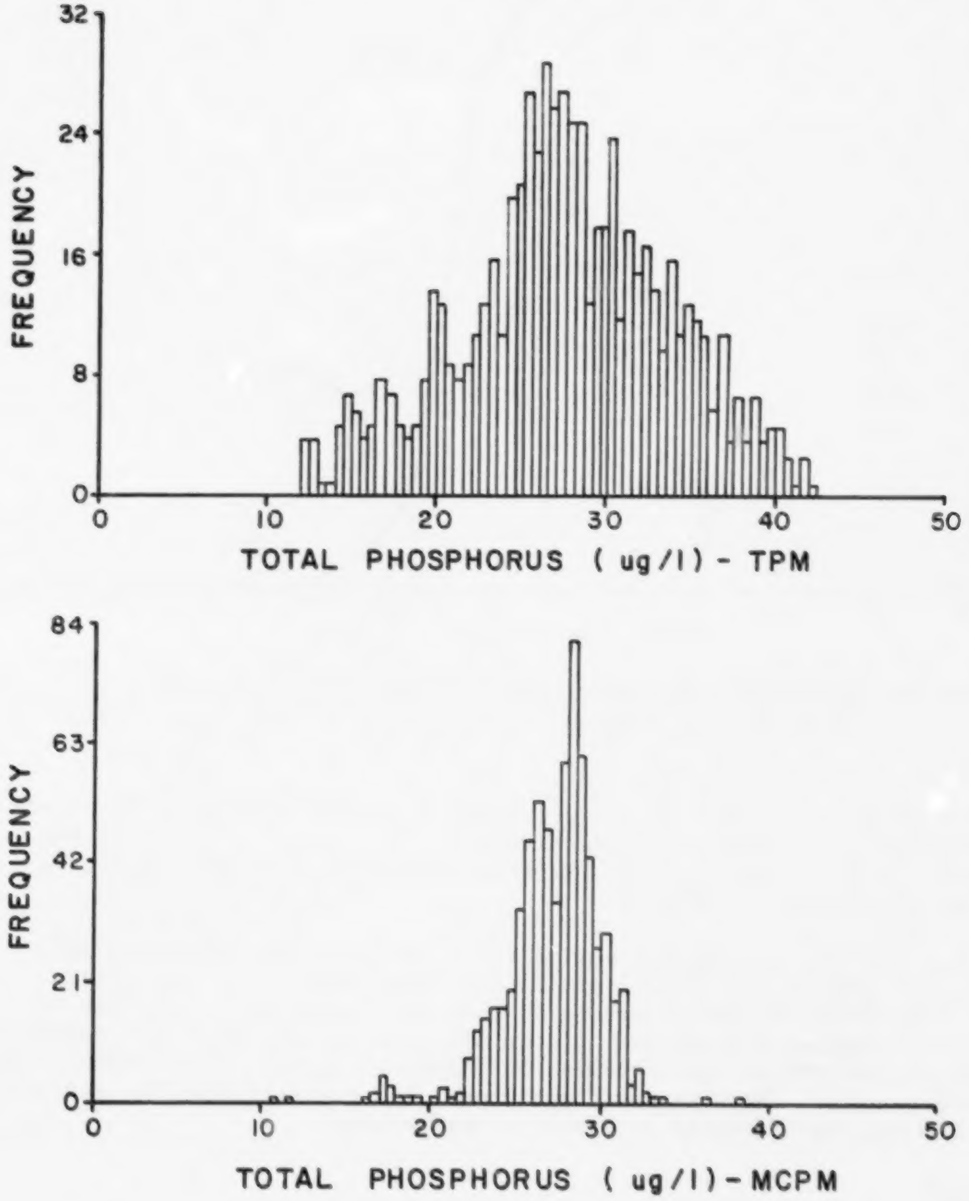


Figure 6a. Frequency Distribution for Time-Variable Simulation - Case I

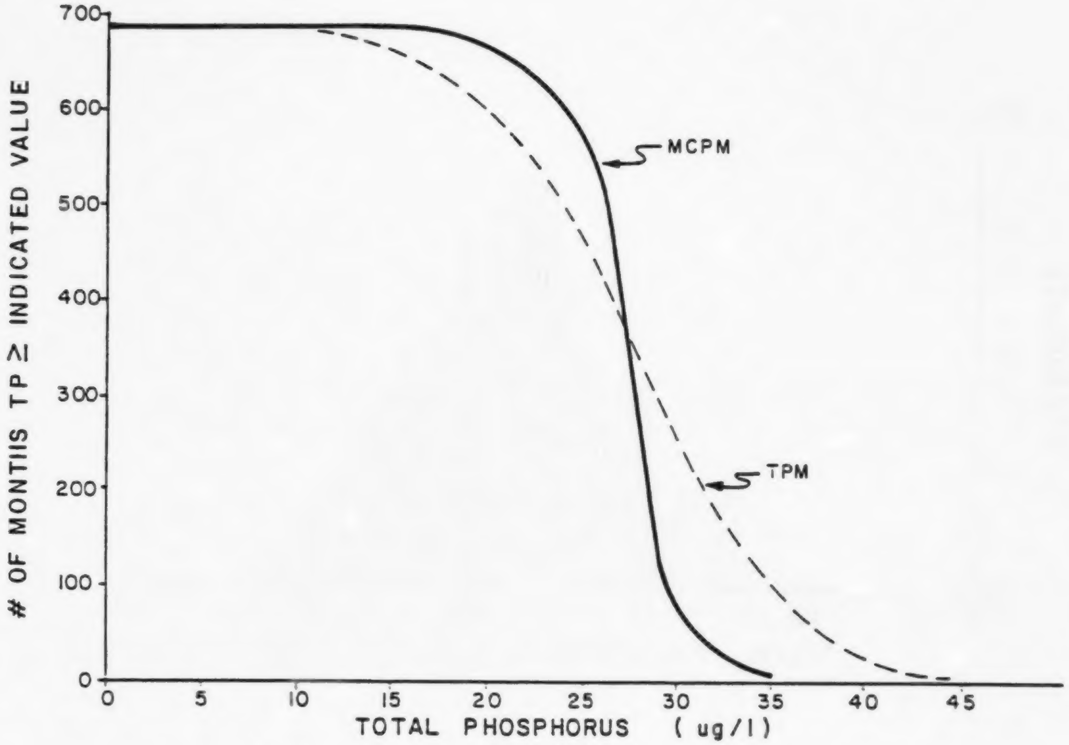


Figure 6b. Cumulative Frequency Distribution for Time Variable Simulation - Case I

ment with the range of 10 - 16 m/yr, a range which was determined from data on predominantly biologically productive eutrophic lakes. This study reveals that for a less productive oligotrophic/mesotrophic impoundment, v_s should be substantially reduced.

Results of the statistical analysis for this case are presented in Table 5, and lead to conclusions similar to those drawn from Case I and Table 3. The distribution of predicted monthly average concentrations is presented in Figures 8.a and 8.b. Again the data are approximately normally distributed, but TPM is characterized by a greater degree of variation as in the previous case. The regression analysis again revealed that the models are predicting significantly different distributions.

DISCUSSION

The analyses presented above raise an important question as to the applicabilityof a simple total phosphorus model for predicting trophic state in impoundments. Models which represent the influence of biological activity on the cycling of phosphorus between organic and inorganic forms include functional relationships for their reaction rate coefficients which express a dependence on various internal or external system parameters. An example

TABLE 4 - $\overline{TP}_{TPM}$ AS A FUNCTION OF v_s - CASE II

v_s	$\overline{TP}_{TPM}$ (ug/l)
1	11.2
2	10.6
3	10.0
5	8.7
8	7.3
10	6.6

of this in the multi-component phosphorus model is the organic phosphorus growth rate term which is a function of nutrient availability. If nutrient concentrations are high, biological uptake and deposition will also be high. The converse is also true. The total phosphorus model with its single reaction rate coefficient does not include this flexibility, and hence as demonstrated, two distinctly different values for v_s were required to obtain the appropriate model response under the two distinctly different phosphorus loading scenarios.

This phenomenon is clearly demonstrated in Figure 9 which is a plot of the time-variable nutrient limitation term. Note that in Case I, values are typically greater than those in Case II corresponding to the reduction in v_s from 9 to 3 m/yr respectively. To further verify this, a third simulation

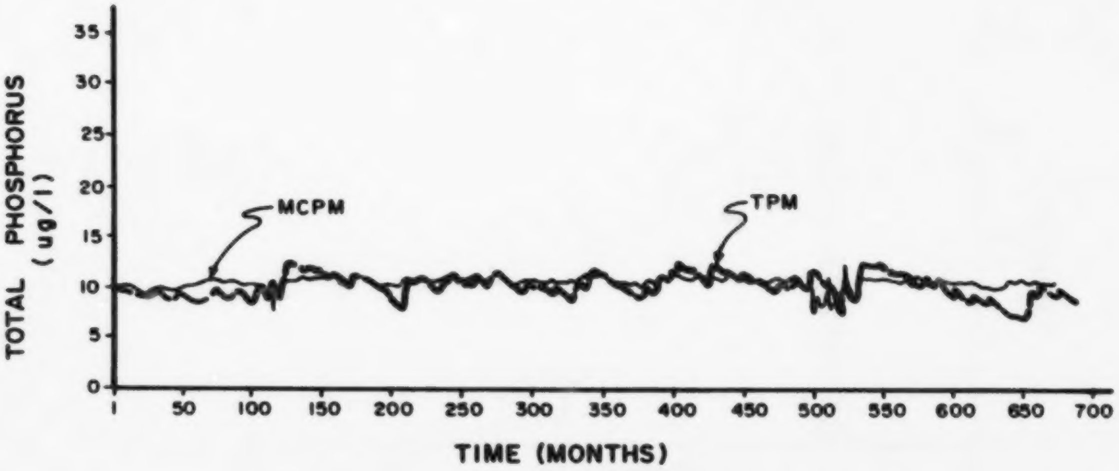


Figure 7. Time Variable Simulation - Case II

(dashed line represents TPM; solid line represents MCPM) was conducted, utilizing conditions identical to Case I, however, the half-saturation constant was increased by a factor of three for demonstration purposes. $\overline{TP}_{MCPM}$ was determined to be 37.4 ug/l, indicating fairly enriched conditions, yet the appropriate value of v_s was found to be 4 m/year. The nutrient limitation term illustrated in Figure 9 provides the explanation. Here again the net organic phosphorus growth rate term is reduced preventing

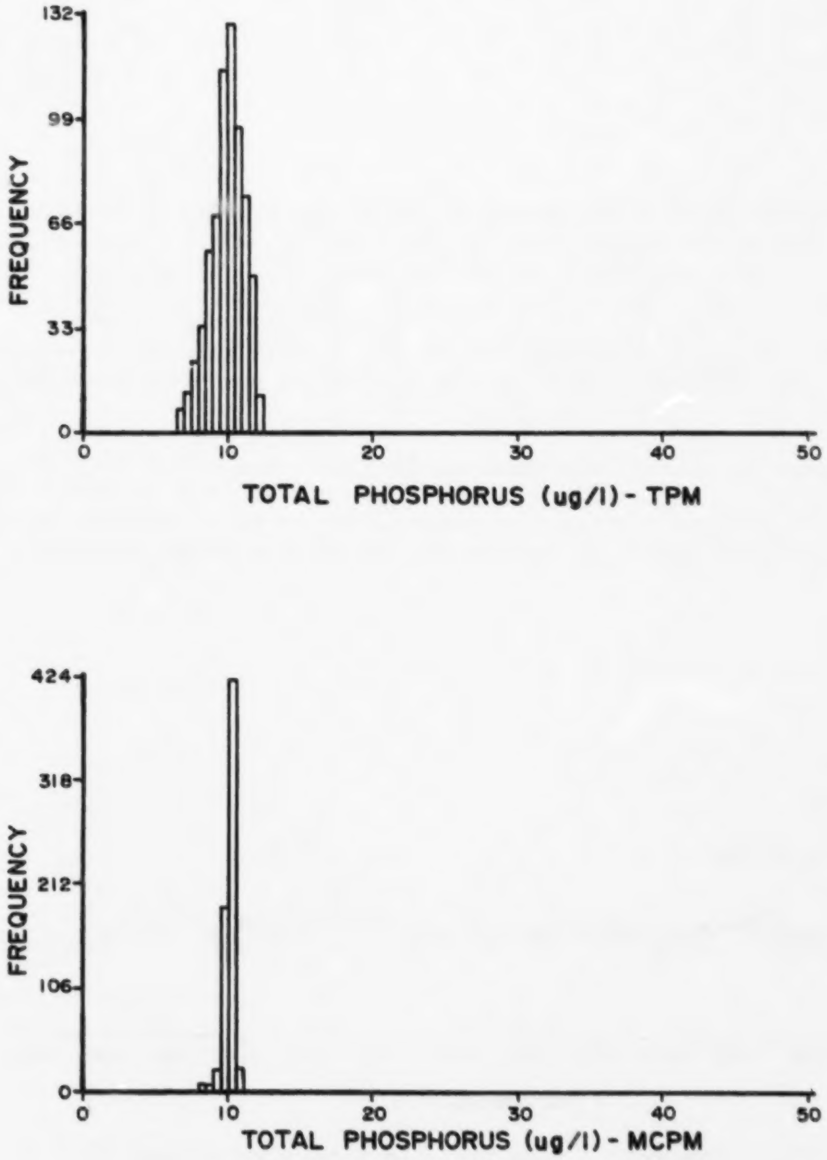


Figure 8a. Frequency Distribution for Time-Variable Simulation - Case II

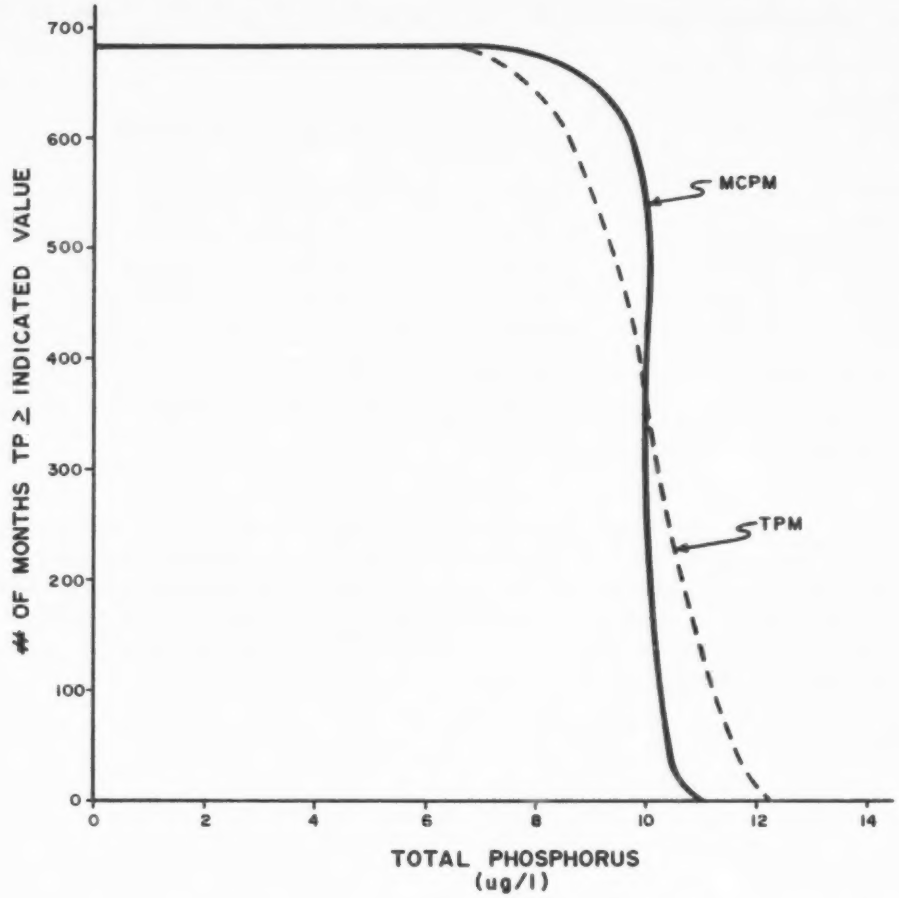


Figure 8b. Cumulative Frequency Distribution for Time Variable Simulation - Case II

the cycling of phosphorus into the organic form with subsequent deposition to the sediments. This study demonstrates the importance of including the strong influence of biological activity to represent the net sedimentation of phosphorus in a model for trophic state simulation. A simpler version does not respond accordingly for time-variable simulation, unless of course v_s is adjusted as demonstrated to be required above.

The nutrient limitation term is also the key to the observed difference between relative error and correlation coefficients for the two cases as reported in Tables 3 and 5. There was significant improvement in Case II as a result of the more constant nature of the nutrient limitation term. This reduces the nonlinearity of the multicomponent phosphorus model and provides better agreement with the linear total phosphorus model.

Another difference which becomes apparent from this study is the consistently higher coefficient of variation observed with the total phosphorus

TABLE 5 - STATISTICAL ANALYSIS OF TIME-VARIABLE SIMULATION - CASE II

Averaging Group	R	RE	Slope	Intercept
Monthly	0.979	0.098	1.55	-5.53
1 year	0.949	0.095	1.7	-6.9
4 year	0.776	0.078	1.64*	-6.48*
10 year	0.374	0.037	1.54*	-5.34*
25 year	0.197	0.020	1.96	-9.53
57 year	0.05	0.005		

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*Indicates value not significantly different than 1.0 for slope or 0.0 for intercept at 95% confidence level.

model in both steady-state and time-variable simulations. This results from the fact that TPM because of its simple biochemical structure, is more strongly dominated by input-output forces, whereas MCPM represents an effective ecological buffering capacity which reacts to changes in external forces. This difference in variance is most clearly demonstrated in the frequency distribution histograms of Figure 6 and 8.

CONCLUSIONS

Several conclusions have been developed from this study. For steady-state simulation, the two models do not differ appreciably in their prediction of total phosphorus. However, for time-variable simulation, the models

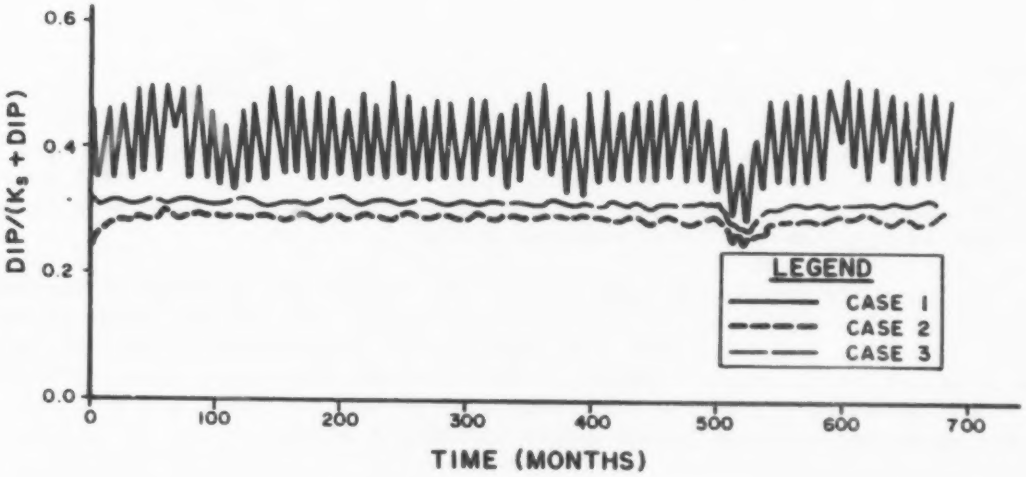

Figure 9. Time-Variable Nutrient Limitation Term

do predict significantly different concentrations. Furthermore, the frequency distribution of those concentrations even over very long term simulation periods also differ. For a given simulation, the total phosphorus model will predict a wider range of concentrations than the multi-component phosphorus model.

It is important to include a functional relationship describing the influence of biological activity in the cycling of phosphorus through its various organic and inorganic forms thereby determining the net sedimentation rate for total phosphorus. The value for v_s is not constant across trophic states. The nonlinearity of the nutrient limitation term is responsible for the difference between model predictions. The relative error between the models decreases as the degree of variation in this term decreases.

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F.D.O.T. Drainage Manual:Why "Drainage"in an Age of "Stormwater Management"?

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Tallahassee, Florida 32301

ABSTRACT

In 1963, the Florida State Road Department issued the first "Drainage Manual". New policy and procedures were added by memorandum and design directives. In the intervening 23 + years, enviromental awareness in general and in Florida in particular increased dramatically. The intervening development of Florida stormwater regulation is traced. When in 1985 a replacement manual was begun, the decision was made to retain the title "Drainage Manual" rather than changing the title to "Stormwater Management Manual". This was not done to indicate that stormwater management was to be disregarded, but to emphasize that drainage, which is critical to transportation, must be provided. An overview of the draft Drainage Manual is presented which demonstrates that proper drainage is good stormwater management. Ineffective or inadequate drainage is inappropriate stormwater management.

1963 FSRD DRAINAGE MANUAL

In Florida the "state of the art" for highway drainage as it existed 25 years ago can be seen by reviewing the 1963 (revised 1967) Florida State Road Department Drainage Manual. A listing of the chapter headings is provided in Table 1. The 1963 manual has only a few tables, equations and figures which are identified in Table 2. The manual relied on the user having a good working knowledge of hydrology and hydraulics and a fundamental hydrologic/hydraulic library upon which to draw. The manual supplied the necessary information to enable the user to utilize reference information to accomplish highway drainage to Department standards.

What was the thrust of the manual? In two words it was runoff conveyance. The methods were simple and straight forward, and not surprisingly, they worked. This can be explained because the manual was

TABLE 1. STATE ROAD DEPARTMENT DRAINAGE MANUAL 1963
TABLE OF CONTENTS

Chapter	Chapter Heading	Pages
1	Introduction	1
2	The Drainage Section	1
3	Engineering Law	10
4	Policies	5
5	Survey and Field Information	3
6	Rainfall and Runoff	15
7	Hydraulic Design of Culverts	4
8	Ditches and Canals	5
9	Hydraulic Design of Bridges (never completed)	0
10	Subgrade Drainage	2
11	High Water Clearance for Pavement Grades	1
12	Storm Sewers	9
13	Checking Plans	6
	TOTAL	62

written by District and Central Office drainage engineers reflecting 35 + year record of the highway drainage experience in Florida.

By 1963, Florida had begun to stir and start the transition from a mostly rural state with a few predominately coastal urban centers into one of the fastest growth states in the U.S. It is safe to categorize the 1960's Florida drainage law as "common law" which permitted upper owners to increase the rate (quantity) of discharge provided the increase did not cause damage. The standard applied was one of reasonableness. This has been described as a modified "look out below" drainage law. By the late 1970's most major urban areas developed regulations and ordinances which limited and defined maximum allowable peak discharges, since the cumulative effect of many "resonable" increases is not necessarily reasonable. The special problem of total volume amount in landlocked watersheds prevalent in north and central Florida's karst topography was not addressed.

The first Drainage Manual required the highway drainage designer to consider future development within a 20 year period but did not allow private connections to the Department's drainage facilities. However, connections were routinely granted provided the applicant demonstrated that the discharge does not come from divirted areas and that the Department's system was designed for the level of discharge. The conveyance system below the Department's outlet has always been evaluated with the extent of the evaluation increasing in recent times. Quality was addressed in general terms. Quality requirements were broad, requiring dischargers to meet State water quality standards. A general interpretation of the earlier requirement was "do not contaminate the stormwater".

TABLE 2. STATE ROAD DEPARTMENT DRAINAGE MANUAL 1963
TABLES , EQUATIONS AND FIGURES

Tables	Equations	Figures
Storm Design Frequency	Rational	Rainfall Stations
Runoff Coefficient	Talbot's Formula	Intensity-Duration Curves
Minimum Culvert Sizes		Time of Concentration for Overland flow
Cleaning Velocities		
Base Clearances		Talbot's Discharge Curve
Manning's "n" Values		Storm Sewer Tabulation Form
Maximum Manhole spacing		Highway Section Check List

THE INTERVENING PERIOD - 1963 to 1985

Prior to 1972, water management legislation had developed on a piecemeal basis. In that year, a comprehensive law has enacted to provide extensive protection and management of water resources throughout the state.

The Florida 1972 Water Resources Act, Chapter 373, Florida Statues, provides a two-tiered administrative structure presently headed at the state level by the Department of Environmental Regulation (DER). DER is the chief pollution control agency in the state and was created from parts of several agencies through Legislative reorganization in 1975. DER's jurisdiction over water pollution control extends to "waters of the state". Practically all dredge and fill activities conducted in areas either in or connected to waters of the state are required to comply with water quality standards. DER regulates discharge or untreated stormwater that could be a potential source of pollution to the state. This regulatory scheme is predominantly qualitative. All stormwater discharges must meet the water quality standards of the class of water body the stormwater actually reaches. Additionally, DER regulates stormwater by requiring retention or retention with filtration systems that allow separation of polluting substances by percolating the water into the ground. DER supervises five regional Water Management Districts designed to provide the diverse types of regulation needed in different areas of the state. These include the previously existing Central and Southern Florida Flood Control District, renamed the South Florida Water Management District, and the Southwest Florida Water Management District. The three new districts established under the Act were the Suwannee River, St. Johns River, and Northwest Florida Water Management Districts.

Most of the thrust of DERs and the Water Managment Districts efforts have been towards water quality. Efforts to address stormwater quantity control were mainly left to local government. Under the present law, municipalities have authority to provide for drainage of city streets and reclamation of wet, low, or overflowed lands within their jurisdiction. They may construct sewers and drains and may levy special assessments on benefited property owners to pay all or part of the costs of such works. Additionally, municipalities have the power of eminent domain to condemn property for these purposes. Thus, they have the means to deal directly with stormwater and surface-water runoff problems. Municipalities may enact floodplain zoning ordinances. Such ordinances may simply require compliance with special building regulations or may exclude certain types of development in a designated floodplain.

Most counties and municipalities have a drainage plan ordinance that requires submittal of a drainage plan for proposed developments. In addition, they commonly require that a drainage impact assessment be prepared and submitted if there is to be a change in the development site. Many local governments have ordinances restricting the amount of surface-water runoff that may be carried by a particular drainage system, or the amount of sediment transported by the runoff. Many local ordinances also incorporate a flood plain regulation element or minimum elevations.

Subdivision regulations relating to surface-water runoff control tend to be more detailed than local government ordinances, and often require submittal of a comprehensive drainage plan, approval of which is often a prerequisite for plat approval. Some regulations include runoff and rainfall criteria to which the proposed drainage system must conform, while others indicate permitted or preferred surface-water runoff control structures and techniques. Other provisions found in subdivision regulations include: a requirement that runoff from paved areas meet certain water quality standards; the encouragement or requirement of onsite retention of runoff; the regulation of grading and erosion control methods; and a monitoring requirement for the discharge of surface-water runoff into lakes, streams, and canals.

The Department of Community Affairs has recently been actively charged with the responsiblity of coordinating growth management in the State, which will reflect on drainage facilities and projected areas growth.

To provide adequate drainage and stormwater management, the regional Water Management Districts work in conjunction with the Departments of Community Affairs and Environmental Regulation to obtain the necessary master planning and overall coordination. The transportation facilities overseen by the Department of Transportation must be recognized and included in the planning and implementation strategy of these conjoining authorities.

1986 FDOT DRAINAGE MANUAL

The Florida Transportation Code establishes the responsibilities of the State, counties, and municipalities for the planning and development of the transportation systems with the objective of assuring development of an integrated, balanced statewide system. The Code's purpose is to protect the safety and general welfare of the people of the State and to preserve and improve all transportation facilities in Florida. The Code sets forth the powers and duties of the Department of Transportation to develop and adopt uniform minimum standards and criteria for the design, construction, maintenance, and operation of public roads.

The policy set forth in the manual provides guidelines for the planning and design of drainage and stormwater management control features for the Department. The guidelines, while positive and effective, are intentionally broad, since no policy statement can cover all contingencies. Its primary purpose is to establish uniform practices and to ensure that the best information available is applied to project conditions. It neither replaces the need for professional engineering judgement nor precludes the use of information not presented in the manual.

Whether the Department must comply with local rules and programs for stormwater management is a question that generates great doubt and confusion. As a general rule, the Department should cooperate and comply with local regulations where such compliance would not be detrimental to the Department's interests or its ability to carry out its responsibilities.

Stormwater management must be a marriage of both quantity control and quality control. This is the direction taken by the proposed Drainage Manual. The manual is presented in three volumes: Policy, Procedures, and Theory.

Volume One sets forth the prevailing policies as they pertain to collecting, receiving and passing stormwater runoff from and through the rights of way. The policies reflect the Department's responsibility to act on the public's behalf in matters of safety, economics, effectiveness, and environmental concerns. A listing of chapters is contained in Table 3(a).

Volume Two provides the most common or usually appropriate procedures required to design the accordance with the policy. Volume Two includes frequently used charts, equations, computational forms and figures extracted from numerous technical publications. Textual material from these publications is not duplicated in its entirety. Discussions on applying the procedures focus on step-by-step calculations which assume that the reader has a working knowledge of theoretical fundamentals and terminology. Desktop technical procedures are presented in the manual. Computer programs of the Department's mainframe computer are summarized. Tables and figures are located at the end of each chapter. A listing of chapters for Volume Two is presented in Table 3(b).

TABLE 3(a) 1986 FDOT DRAINAGE MANUAL VOLUME 1 - POLICY

Chapter	Chapter Heading		Pages
1	Introduction		2
2	Organization		8
3	Drainage Law		18
4	Policy Guidelines		21
	Appendix		9
		TOTAL	58

TABLE 3(b) 1986 FDOT DRAINAGE MANUAL VOLUME 2 - PROCEDURES

Chapter	Chapter Heading	Pages
1	Introduction	3
2	Planning and Facility Location	9
3	Flood Plain Encroachment and Risk Evaluation	25
4	Data Collection	39
5	Stormwater Hydrology	68
6	Roadway Grades	5
7	Open Channel Hydraulics	40
8	Culvert Hydraulics	76
9	Bridge and Bridge Culvert Hydraulics	25
10	Storm Sewer	40
11	Culvert Material Selection	23
12	Gutter, Inlet and Pavement Hydraulics	32
13	Retention/Detention Hydraulics	20
14	Stormwater Pumping	12
15	Subsurface Drainage	32
16	Shore and Bank Protection	21
17	Erosion and Sediment	30
18	Computer Programs	14
19	Standard Indexes and Special Details	30
20	Permitting	39
	TOTAL	573

Volume Three provides background information on drainage fundamentals, supplementing the procedures presented in Volume Two. The focus of the contents is on theoretical background. Not all chapters contained in Volume Two are included. A listing of the chapters is presented in Table 3(c).

The manual has increased in size due mainly to an attempt to provide most of the more widely utilized engineering materials. Stormwater quality management requirements and recommended procedures are given chapters rather than lines. There is also a difference in quantity requirements that must be emphasized. In recoginzation of recent adverse court decisions the Department has altered its design concepts significantly. Increases in peak stormwater discharge rates by upland owners will not be accepted or designed for unless it is part of or provided for in a comprehensive local plan which is substantially complete.

Volume One, Chapter 4 includes permitting both by and with the Department. A discussion on the consideration of future development in the design is presented.

Volume Two, Chapter 2 deals with floodplain encroachments, establishing the level of impact, detailing agency coordination, planning and permitting at the location planning phase. Chapter 3 identifies various highway

TABLE 3(c) 1986 FDOT DRAINAGE MANUAL VOLUME 3 - THEORY

Chapter	Chapter Title	Pages
1	Introduction	3
2	Surface Hydrology	45
3	Subsurface Drainage	18
4	Open Channel Hydraulics	23
5	Culvert Hydraulics	14
6	Bridge and Bridge Culvert Hydraulics	24
7	Storm Sewer Hydraulics and Hydrology	8
8	Gutter, Inlet and Pavement Hydraulics	20
9	Retention/Detention Hydraulics	12
10	Erosion and Sediment Control	17
11	Shore and Bank Protection	19
	References	11
	Glossary	25
	TOTAL	239

activities in floodplains giving the level of studies required by Drainage for each activity. Chapter 5 discusses various hydrologic procedures, often required to satisfy permit requirements of other agencies as well as Department design needs. Chapters 6,7 and 8 are hydraulic procedures used to establish highway grades, open channel flow and culvert requirements. Chapter 9 provides information required for bridges and bridge culverts, including coordination requirements with the local communities when Federal Emergency Management Agency (FEMA) regulated floodways are involved. Chapter 13 provides design procedures for retention and detention design both to reduce outfall requirements and to conform with permit requirements. Flood routing procedures, filtration calculations for detention with filtration and landlocked basin retention requirements are presented. Chapter 15 provides procedures for subsurface drainage, including exfiltration (french) drains and drainage well designs. Chapter 16 provides information for shore and bank protection. Chapter 17 contains erosion and sediment predictive methods and control procedures both temporary and permanent. Standard details and indexes used by Drainage are explained in Chapter 19. Permitting activities with regulatory agencies as well as permits issued by the Department in drainage or stormwater related matters are explained in Chapter 20.

Most of the procedures in the manual are obtained from widely available sources, particularily the U.S. Department of Transportation, Federal Highway Adminstration (FHWA).

The manual will be updated annually. Updated material will be furnished to purchasers of the manual at no charge provided that the manual owner's address is current. The manual is being given a final internal review prior to printing. It is anticipated to be available by Fall 1986. Its purchase price has not been established. If you would like to get on a "reserved" pre-publication list you may do so by either writing or telephoning and requesting to be put on the list. Requests should be directed to the State Drainage Engineer, Florida Department of Transportation, Mail Station 32, 605 Suwannee Street, Tallahassee, Florida 32301; Telephone (904) 487-1700. Persons who are on the list will be advised of the price and availability and would be guaranteed a copy from the initial printing.

I hope the preceeding presentation has provided insight into the changing needs which the new Drainage Manual is addressing. The need for safe, well-drained and available transportation facilities has not changed, but the technique required to meet a new awareness of the need for quanity and quality stormwater management has changed significantly. The Department is dedicated to preserving and enhancing the unique environmental geographical area known as Florida while providing for its transportation needs.

The Department should be considered as a "developer" of transportation facilities much the same as a residential site developer or a commercial site developer. As such the Department should support and conform to regional and local comprehensive stormwater plans rather than establish such plans. The primary need of the Department remains adequate drainage. Required stormwater management practices should reflect the procedures developed and required by those agencies primarily responsible for the entire basin.

The work described is this paper was not funded by the U.S. Environmental Protection Agency and therefore the contents do not necessarily reflect the views of the agency and no official endorsement should be inferred.

APPLICATION OF THE OTTSWMM MODEL
FOR A RELIEF SEWER STUDY IN LAVAL, QUEBEC

by

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J.C. Pigeon (2)
J.R. Noiseux (3)

ABSTRACT

The OTTWSMM (Ottawa University Storm Water Management Model) model is an expanded version of EPA - SWMM which accounts for overland flow, inlet restriction and surface ponding. The model was recently applied in the relief sewer study on an area with an old combined sewer in Laval, a municipality in the Montreal Urban Community.

The first part of the paper describes briefly the area, flooding problems and constraints related to the interceptor and high levels in the receiving water body.

The second part of the paper describes the solution developed by means of the inlet control analysis and gives an economic comparison with more additional alternatives.

The last part of the paper presents some considerations on the use of the model and its interface with a simple screening technique.

INTRODUCTION

Severe flooding problems in the city of Laval prompted the municipal authorities to request a full scale investigation. This was undertaken by Les Consultants Dessau Inc. and the proposed design is presented in this paper.

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The city of Laval is the second largest city in the province of Quebec, Canada and has a population of 300,000. It is situated on an island surrounded by Des Mille-Iles and Des Prairies rivers. The undertaken investigation deal with a particular watershed currently served by a combined system.

DEFINITION OF THE PROBLEM

FLOODING PROBLEMS

The Montmorency-Brien watershed, located in Laval, Quebec, has a drainage area of 102 hectares. A single family housing development has led to an imperviousness of 35%. This urbanized area is currently drained by a combined sewer system that appears to be insufficient to handle a 10 year flow.

In order to determine the extent of flooding, a study was performed in which citizen complaints were evaluated. The following trends were evident:

- Flooding occurs all over the watershed as a result of insufficient pipe capacity (collector and laterals)

- Spring and Summer are the critical flooding periods with lower elevation areas being the most affected. The frequency of flooding has been known to range from none to the three times per year. The average return period of flooding for the whole watershed is around two years.

- Most complaints were for basement flooding as a result of combined sewer surcharge backing up into the houses.

It should be pointed out that the study was based on actual complaints registered at the City Hall. In reality, the number of flooding incidents is somewhat higher.

PRELIMINARY INVESTIGATIONS

A computer simulation of the existing pipe network indicated that 90% of the existing collector pipes as well as 60% of the lateral pipes do not meet the up-dated design criteria required by the City of Laval. The collector average capacity corresponds to a two year storm while the lateral pipes indicated a very diverse capacity to handle storm generated flow (3 times in a year to one in 25 years). These are definitely not within the 10 year return period design restraints as specified by the city.

In an attempt to better comprehend the flooding problems, Spring and summer conditions were individually simulated. Spring conditions are characterized by a lower rainfall (3 times less than summer conditions) and a high water level of the receiving body as a result of snowmelt. It is indicative that the outfall pipe is found to be completely submerged. As a result of the higher water level in the receiving water, flooding conditions are more severe in the depressed areas, which have a lower potential energy. This was reflected in the citizen complaints. During the summer months, the water level is at the half-point mark in the outfall pipe even though the combined sewer is subject to intense rainfall.

As a result of the preliminary analysis, it was felt that due to the existing circumstances, all pipes should be individually analysed. For these purposes, the OTTSWMM model (Ottawa storm water management model) was used in connection with the surcharge sub-model EXTRAN (Extended transport). In this way, the influence of the variations of the river level and the very low slope of the existing combined collector were accounted for.

OPTIMUM SOLUTION

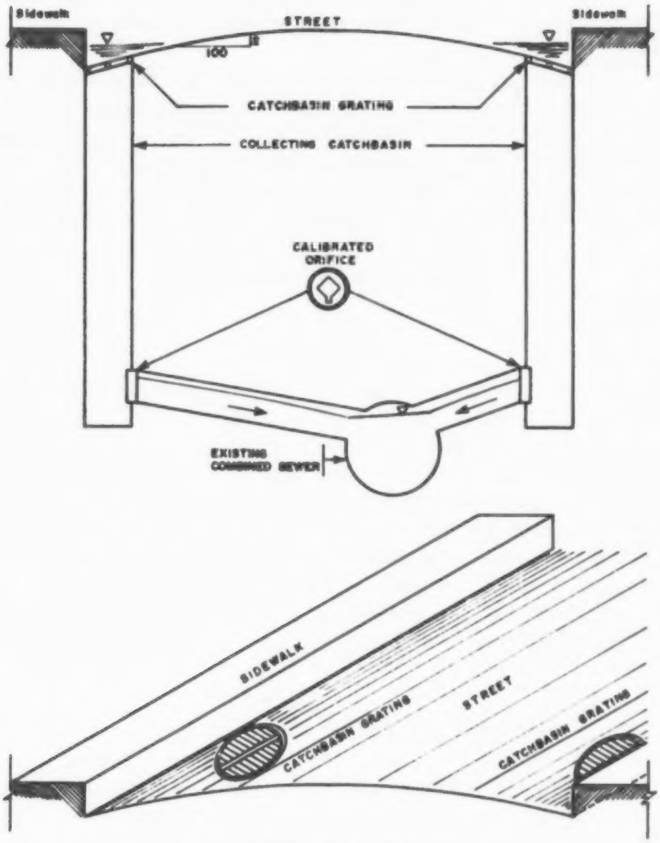
DESCRIPTION

The traditional approach to rectify the flooding problems of the Montmorency-Brien watershed would have called for the twinning of 60% of the lateral pipes. Due to space limitation twinning could not be applied to the collector pipes hence almost 90% of them would have to be replaced by higher capacity pipes. The implementation of this programm would have led to extensive excavations in the majority of the existing roads. Furthermore, due to topographic peculiarities, a combined sewer of 3,000 mm diameter at a depth of 9 meters would have to be installed at a location. Apart from the obvious economic and severe inconvenience repercussions this approach would involve, it would provide only a limited protection against flooding. (1 in 10 years)

A solution was developed in order to reduce the twinning of the existing sewer system and also provides for a higher protection against flooding. This system utilizes inlet control techniques to reduce the flow entering in the existing system (as indicated in figure 1). Calibrated orifices are installed in the catchbasins in order to restrict the water flow entering in the combined sewer. This keeps the hydraulic grade line below basement levels and consequently, avoids the possibility of basement flooding. This solution uses the streets to carry the water flow exceeding the pipe capacity. Hence the streets are actually acting as open channels, in rare storm events, transporting the excess flow which is subsequently captured by new storm sewers. Figures 2 to 5 outline typical cases of actual conditions and how these will be altered by the implementation of the proposed solution.

The present lateral pipe and collector could not accomodate a 10 year flow. In the proposed system (figure 2b), use of inlet control devices allow for surface runoff to be collected at a lower point by a proposed storm sewer collector. This method avoids twinning of the lateral pipe.

In figure 3a, a low point is located upstream of the existing collector. Street ponding was not a suitable solution due to excessive volume of flow and traffic restrictions. In the proposed system, inlet restrictions are installed in the latter part of the street. A storm collector is installed to accomodate the increased flow to the low point. This eliminates twinning of the latter part of the lateral pipe.


Sidewalk
STREET
Sidewalk
100
2
CATCHBASIN GRATING
COLLECTING CATCHBASIN
CALIBRATED ORIFICE
EXISTING COMBINED SEWER
SIDEWALK
CATCHBASIN GRATING
STREET
CATCHBASIN GRATING

INLET CONTROL

FIGURE: 1

A similar problem as the one shown in figure 3. The solution (figure 4b) involved the installation of an additional lateral pipe and collector nearer to the ground surface. Thus by this technique overloading of the downstream system was avoided.

In a particular location, it was deemed necessary to install restrictions in the catchbasins to direct the flow in a shallow storage. Storm water was stored and gradually released into the system at a rate which allowed for its accomodation by the existing network. While this approach allowed for keeping the lateral pipe and collector intact, it was difficult to install the storage due to espace limitations.

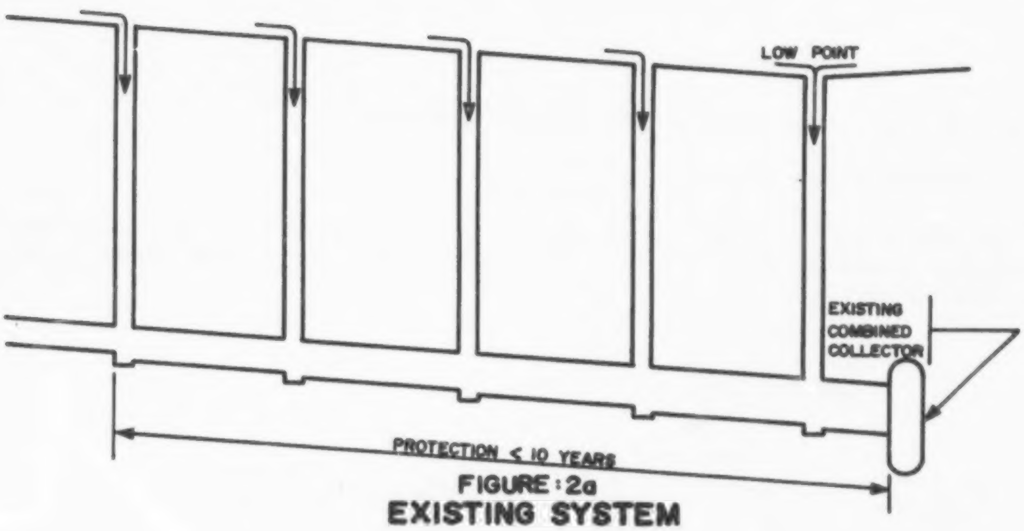

LOW POINT
EXISTING COMBINED COLLECTOR
PROTECTION < 10 YEARS

FIGURE: 2a
EXISTING SYSTEM

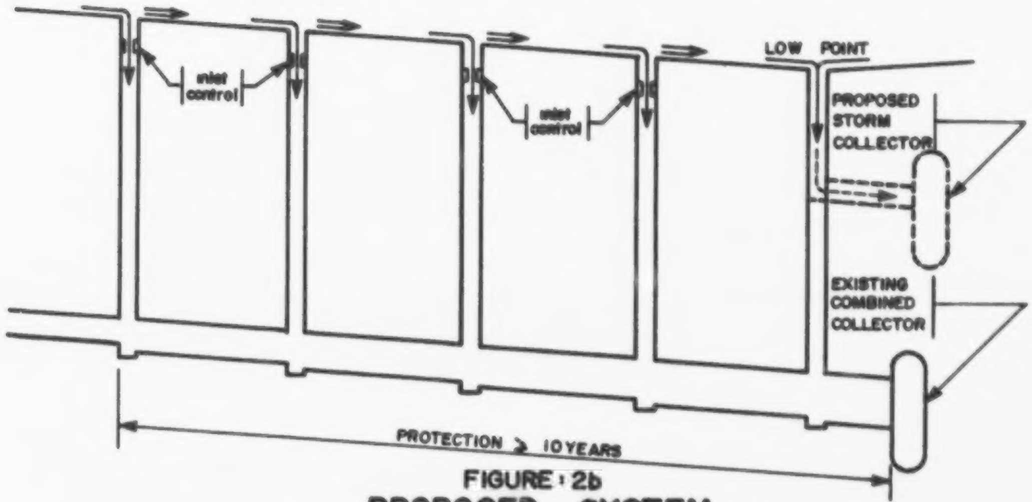

LOW POINT
inlet control
inlet control
PROPOSED STORM COLLECTOR
EXISTING COMBINED COLLECTOR
PROTECTION ≥ 10YEARS

FIGURE: 2b
PROPOSED SYSTEM

DATA INPUT AND ANALYSIS

In order to implement the proposed solution, the following input parameters are required:

- Tributary area
- Elevation of high and low points
- Street longitudinal inclination

- Curb or sidewalk elevation
- Catchbasin grating specification (figure 6)
- Actual pipe capacity

It is pointed out that flow generated by impervious areas directly connected to the system will not flow on street. This was incorporated in all subsequent analysis.

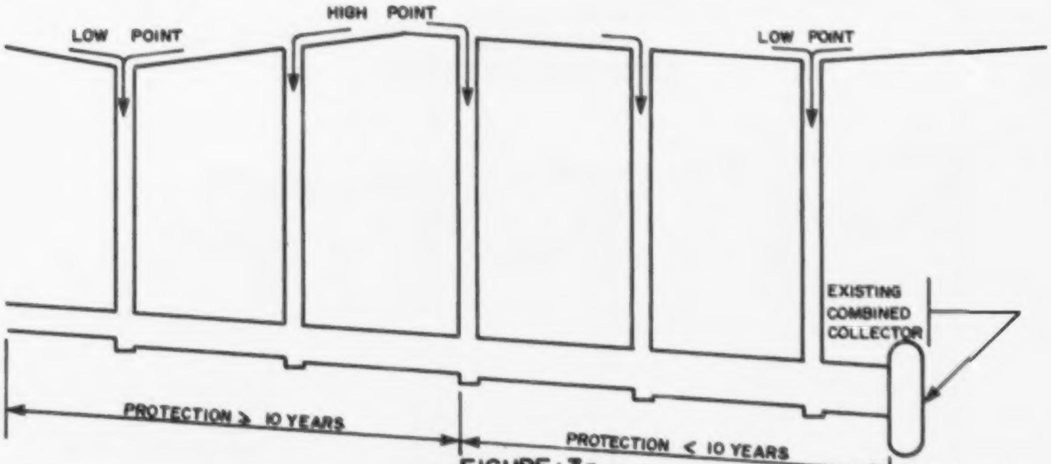

LOW POINT
HIGH POINT
LOW POINT
EXISTING COMBINED COLLECTOR
PROTECTION ≥ 10 YEARS
PROTECTION < 10 YEARS

FIGURE : 3a
EXISTING SYSTEM

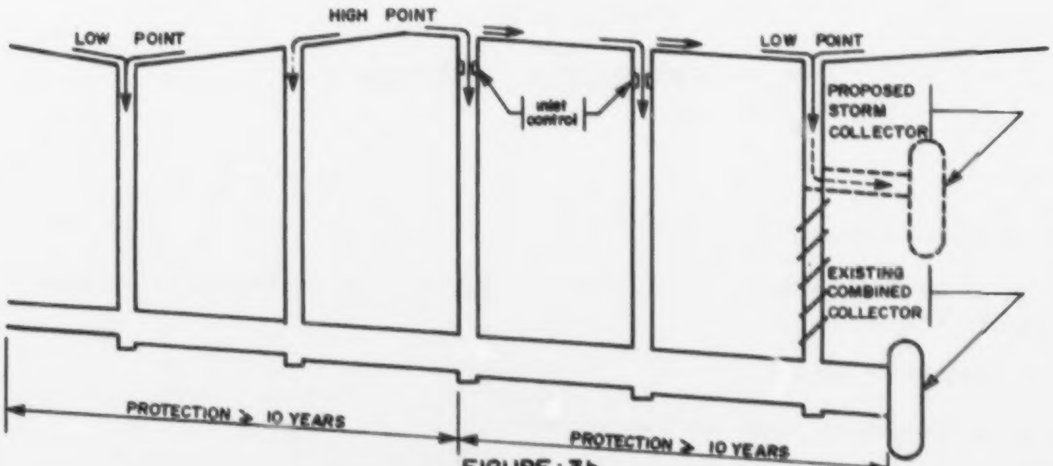

LOW POINT
HIGH POINT
LOW POINT
inlet control
PROPOSED STORM COLLECTOR
EXISTING COMBINED COLLECTOR
PROTECTION ≥ 10 YEARS
PROTECTION ≥ 10 YEARS

FIGURE : 3b
PROPOSED SYSTEM

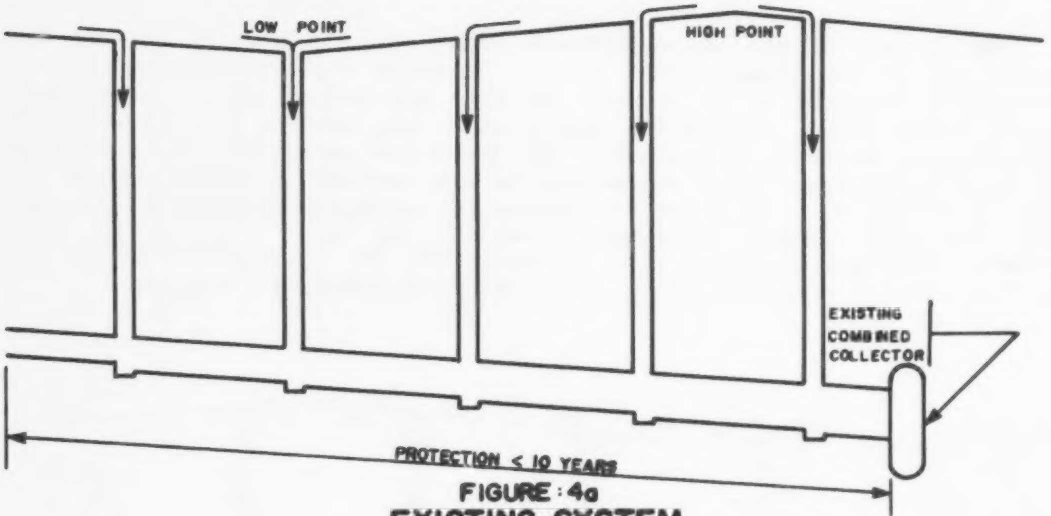

LOW POINT
HIGH POINT
EXISTING COMBINED COLLECTOR
PROTECTION < 10 YEARS

FIGURE : 4a
EXISTING SYSTEM

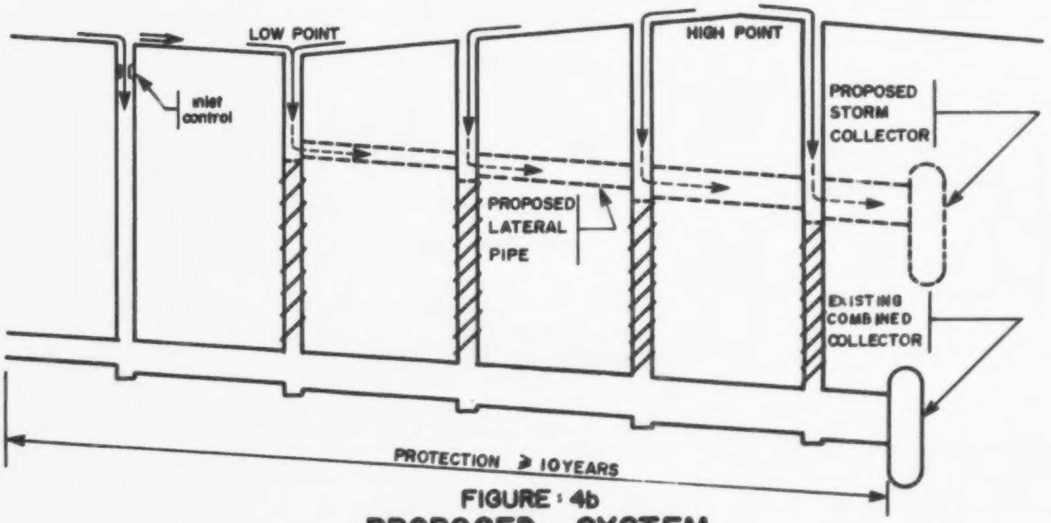

LOW POINT
HIGH POINT
inlet control
PROPOSED STORM COLLECTOR
PROPOSED LATERAL PIPE
EXISTING COMBINED COLLECTOR
PROTECTION ≥ 10 YEARS

FIGURE : 4b
PROPOSED SYSTEM

A detail analysis was undertaken to predict the extent of necessary measures to alleviate the flooding problem in the Montmorency-Brien watershed. It was necessary to perform this analysis pipe by pipe, street segment by street segment and catchbasin by catchbasin. The following were evaluated:

- Flow in the existing combined sewer
- Flow in the proposed storm sewer
- Flow on all the street segments and corresponding depths of flow in order to avoid flooding in shallow curb areas.

The purpose of the above-mentioned detailed hand-analysis was the determination of the optimum inlet control location and the development of a preliminary "sizing" of the pipes. We used the OTTSWMM model (Ottawa University storm water management model) with the results of the detailed hand-analysis as input parameters. The model was manipulated to generated and "cut" hydrographs which correspond to existing and proposed systems (as showm on figure 7). In order to compute the surcharge imposed on the systems, the model EXTRAN (Extended transport) was used in connection with OTTSWMM. This was necessary so as to accomodate the fluctuations of the river flow as well as low gradient of the existing combined collector.

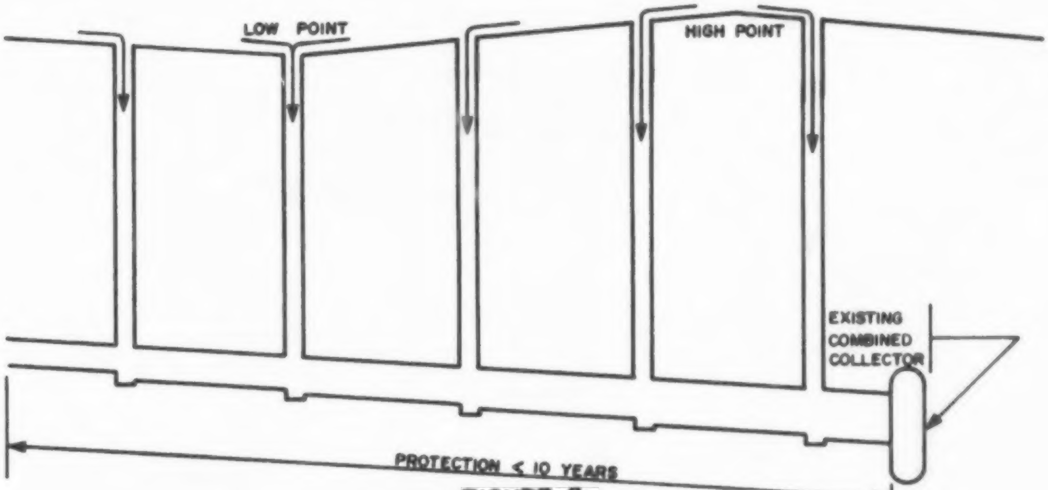

LOW POINT
HIGH POINT
EXISTING
COMBINED
COLLECTOR
PROTECTION ≤ 10 YEARS

FIGURE: 5a
EXISTING SYSTEM

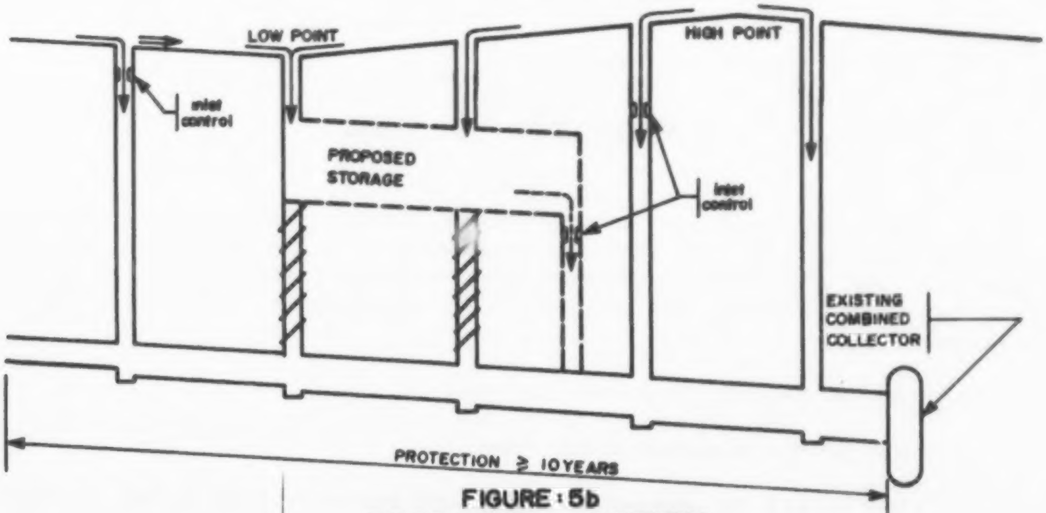

LOW POINT
HIGH POINT
inlet
control
PROPOSED
STORAGE
inlet
control
EXISTING
COMBINED
COLLECTOR
PROTECTION ≥ 10 YEARS

FIGURE: 5b
PROPOSED SYSTEM

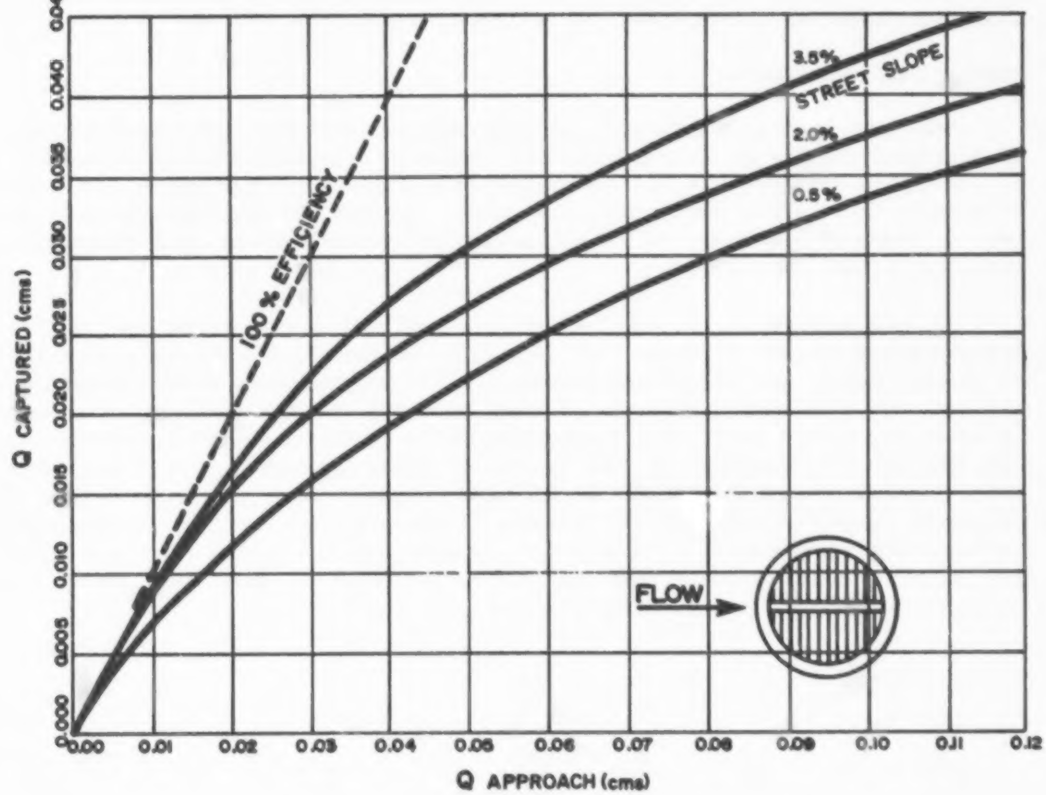
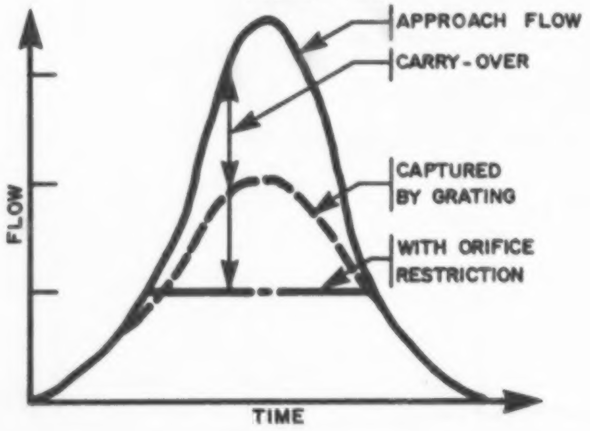


FIGURE : 6a



APPROACH FLOW
CARRY - OVER
CAPTURED BY GRATING
WITH ORIFICE RESTRICTION
FLOW
TIME

FIGURE : 6b

HYDRAULIC PERFORMANCE OF CATCHBASIN

It is important to follow up this analysis by monitoring of the field implementation of the recommended alterations to the system.

GLOBAL IMPLEMENTATION

The global implementation of the suggested solution required several alterations to the existing combined system. A shallow storm sewer collector of 1,650 mm diameter at the outlet was proposed. For space limitations, some wide rectangular sections were used. It was thought appropriate to select a shallow depth so as to minimize excavation costs and disruption of other utilities systems.

It was necessary to completely block 88 catchbasins and to install inlet controls in 40 of them. In the City of Laval, catchbasins are located at almost twice the frequency compared to a neighbouring municipality. Hence, it was possible to block as many without serious repercussions. Two hundred and fifty-four (254) catchbasins of the existing system had to be cut-off and reconnected to the proposed storm system. The reason for the large number of reconnected catchbasins is that the proposed laterals and collector pipes intersect low points where a large number of these catchbasins were situated. A larger than usual special inlet was installed at low points locations in order to discourage surface ponding. The new grating was directly connected to the new storm system.

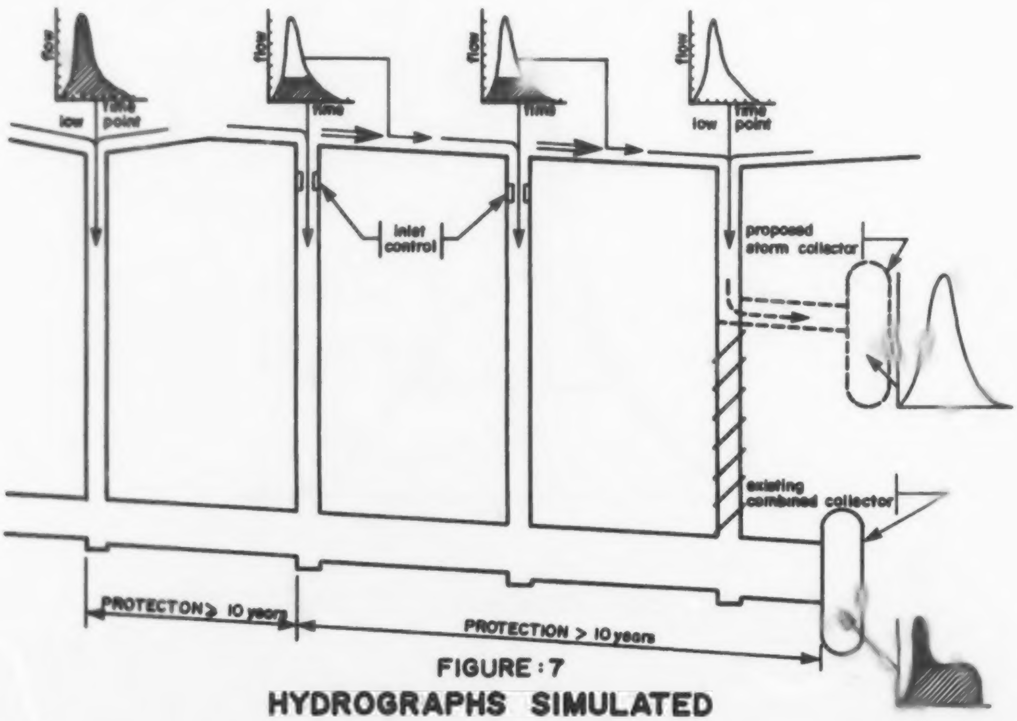

flow
time
low
point
flow
time
flow
time
flow
time
low
point
inlet
control
proposed
storm collector
existing
combined collector
PROTECTON> 10 years
PROTECTION > 10 years

FIGURE : 7
HYDROGRAPHS SIMULATED

As a result of the detailed analysis, four street intersections had to be reprofiled and sidewalk lengths totaling about 60 m had to be built. In this way, runoff flow was prevented from intruding into garages located below the street levels. Furthermore, three (3) new combined lateral piped had to be installed so as to accomodate the flow generated by the impervious areas directly connected.

MODEL SELECTION

It is evident that there is a need for selecting the most appropriate model for a given set of conditions. The opinion of the authors is that simple models should be used in order to better evaluate the prevalent conditions and allow for the selection of the most appropriate sophisticated model for the final design. This approach carries the benefit of reducing the necessary man-hour and computer time.

In this study, the IMPRAM model (Improved rational method) was employed to define the actual problems. Preliminary results prompted the use of OTTSWMM and EXTRAN models for the final analysis.

DISCUSSION AND CONCLUSION

A need to improve the current combined system serving the Laval municipality prompted this study. A novel technique was used by which the existing system was relieved by means of storm sewers and catchbasin inlet controls. This led to the development of a mostly separated system with considerable advantages:

a) Reduction of the length of proposed new pipes from 10 km projected by traditional design techniques to 2,8 km.

b) The installation of the shallow storm sewer will not interfere with other utility connections and will be less prone to river fluctuation influences.

c) As the proposed system is not directly connected to the residences, flooding by surcharge will not be a problem even if the hydraulic grade line reaches the ground surface.

d) Inlet controls keep the hydraulic grade line (of the existing system) below basement levels while still use the existing collector to its full capacity.

e) Implementation of the proposed system will restrict the volume as well as the frequency of combined overflow into the river by about 65%.

f) A traditional approach to this problem was estimated to cost around 7.5 million $. This proposed solution is projected at 4 million $.

ACKNOWLEDGEMENTS

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APPLICATION OF INLET CONTROL DEVICES AND DUAL DRAINAGE MODELLING FOR NEW SUBDIVISIONS

by

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ABSTRACT

Concern over basement flooding has led to the application of a new technique in stormwater management practice, known as inlet control. Major storms have often caused basement flooding when the sewer pipes have surcharged. A traditional, but costly remedy to this problem involves resizing the sewer pipes to handle the higher flows. A new method is to control the flow into the sewer system and thus reduce the occurrence of basement flooding. This is achieved by applying the dual drainage concept and installing inlet control devices in the catchbasins.

This paper summarizes the results of a study conducted for the Canada Mortgage and Housing Corporation that evaluated the performance of three commercially avaialable inlet control devices. The hydraulic operation of the units was tested with a physical model using debris laden water. The study recommended that a minimum size for the orifice type flow regulators be between 14 and 20 lps.

Numerous applications of this concept have been made in the United States and Canada in connection with relief sewer studies. In addition, the concept is being applied in new subdivisions where significant cost savings may be obtained by designing the pipes to be compatible with the level of inlet control.

*presently with CCL Consultants Inc.

INLET CONTROL DEVICES AGAINST SEWER SURCHARGES

The initial objective of storm sewers, traffic convenience, has been replaced by property protection against flooding. This was assured by a judgemental selection of design storm return periods that vary from 5 years (Metro-Toronto) to 100 years (Dallas). After a major storm that caused damage by basement flooding, the traditional solution was to increase the design return period or modify the inlet time in the rational method.

The choice of the municipal engineer used to be between an expensive increase of storm sewer sizes or risk flooding during the life span of a dwelling. In the recent past the problem has become more critical with basements being used for family or recreation rooms in modern homes, damage from flooding is more severe.

One of the most promising solutions to this dilemma was found to be the application of the dual drainage principle in conjunction with inlet control devices (Wisner and Hawdur, 1984). The principle of this solution, which eliminates surcharge without pipe size increases, is shown in Figs. 1 and 2 for new developments and Fig. 3 for old developments with points. This solution is now mandatory in many Canadian municipality where it is designed with the OTTSWMM model. OTTSWMM (The Ottawa University SWM Model) is a modified version of the EPA SWMM-EXTRAN model which accounts not only for conduit but also for overland flow, ponding and inlet controls. A schematic of the models operation is shown in Fig. 4 (Kassem, 1983, Wisner, 1982). The Appendix gives a brief description of OTTSWMM which is available for mainframe and microcomputers. Development of OTTSWMM was made by the IMPSWM (Implementation of Storm Water Management) program at the University of Ottawa. This program is based on a co-operation between IMPSWM researchers and participants from engineering firms and municipalities. The research varies from modelling and experimental studies to review of policies, public attitudes and economics of stormwater management.

Many projects in Southern Ontario have used the dual drainge-inlet control principle in new subdivisions. In these cases the maximum flow accepted in the storm sewer and controlled by ICD's (inlet control devices) varies from 28 lps to 40 lps per catchbasin (Table 1). These limits were selected to eliminate sewer surcharge for a 100 year storm with sewer sizes designed for free surface flow at a 5-year design storm. One of the main reasons for this ICD limit was the lack of information on the performance of various available types of ICD's and concerns regarding clogging. Relief sewer projects in the U.S.A. for older systems used lower limits such as 10 lps per catchbasin or even less. Some of these designs were done with a proprietory model (Donahue 1982), others have been developed without a detailed analysis of street flow depths (Pisano, 1982).

A study co-ordinated by NOVATECH ENGINEERING with co-operation from ANDREW BRODIE ASSOCIATES and IMPSWM in 1985, examined the operation of ICD's and the economic and operational implications of various levels of ICD control. The study was sponsored by the Canadian Housing and Mortgage Corporation. This paper presents briefly some of its findings and conclusions.

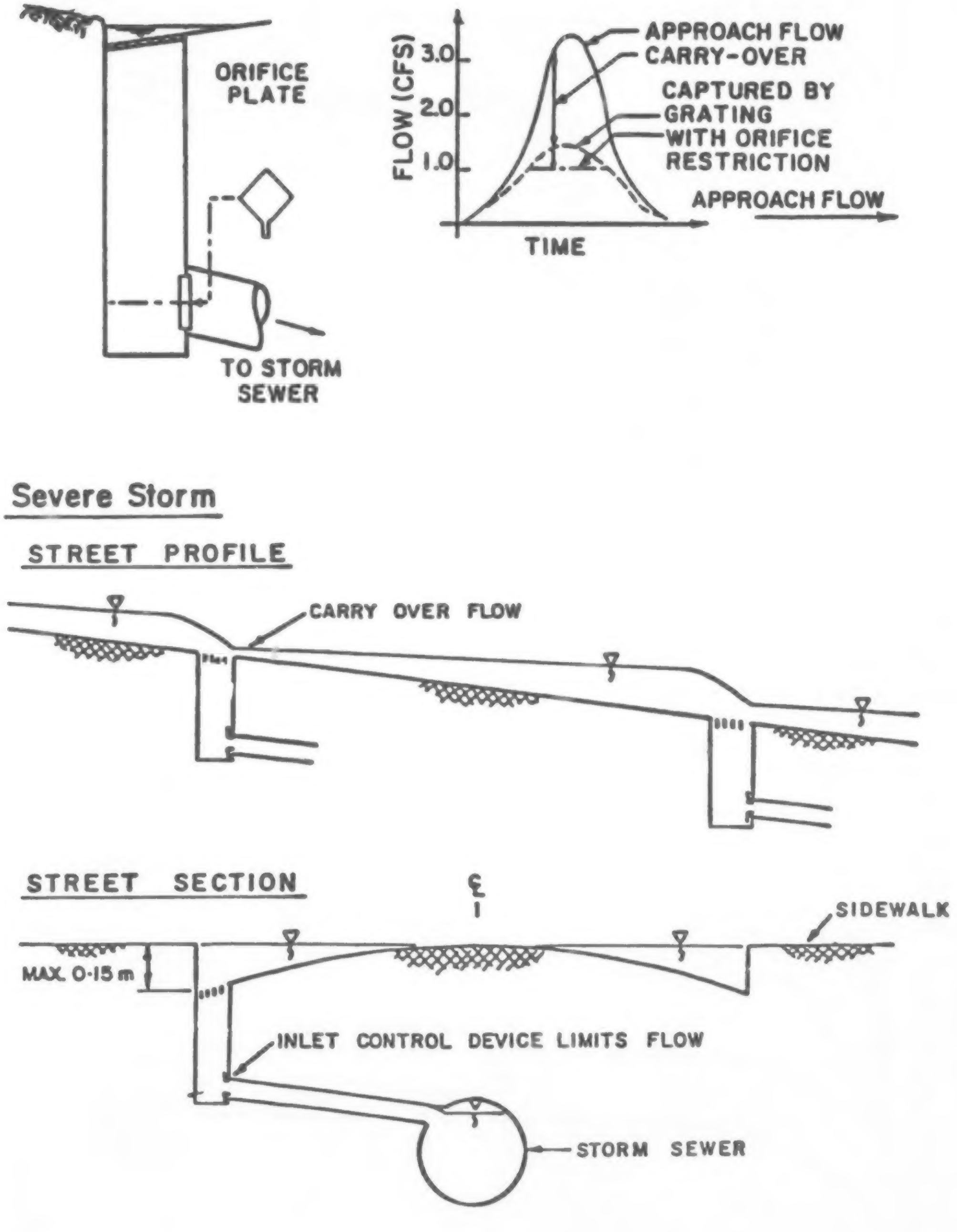

ORIFICE PLATE
TO STORM SEWER
FLOW (CFS)
3.0
2.0
1.0
TIME
APPROACH FLOW
CARRY-OVER
CAPTURED BY GRATING
WITH ORIFICE RESTRICTION
APPROACH FLOW
Severe Storm
STREET PROFILE
CARRY OVER FLOW
STREET SECTION
℄
SIDEWALK
MAX. 0·15 m
INLET CONTROL DEVICE LIMITS FLOW
STORM SEWER

Figure 1 Principle of Inlet Control

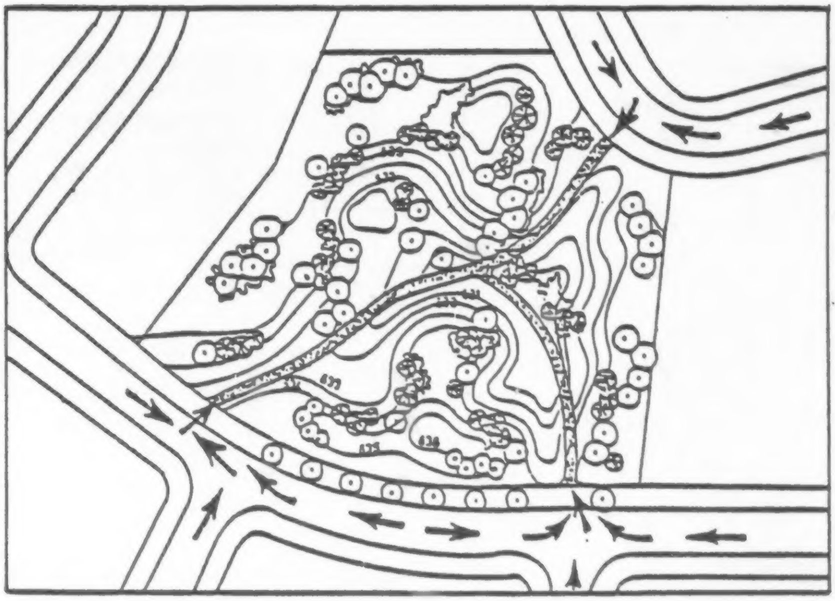


FIG. 2a

LOCATION OF LOW POINTS AND LANDSCAPING OF (DEPRESSED) PARK AREA FOR RUNOFF DETENTION.

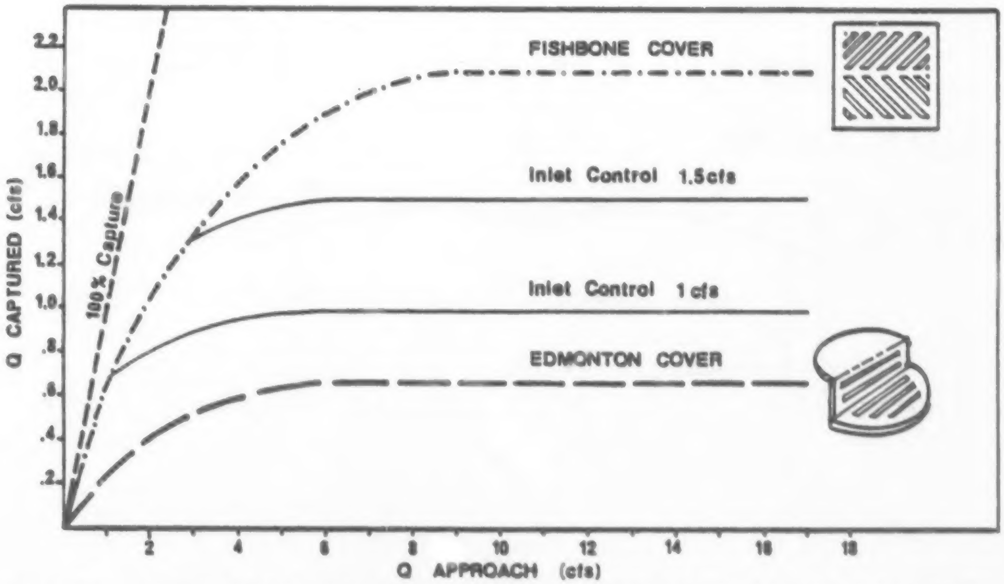


FIG. 2b HYDRAULIC PERFORMANCE OF CATCHBASIN COVERS

EXPERIMENTAL RESULTS

A comparison of several commercial types of ICD's was conducted in the field in Skokie, Illinois (Donahue, 1984). Field observations may confirm if an operation is acceptable or not, but will not explain the hydraulic performance and clogging mechanism. Critical rainfall events are rare, and difficult to monitor. Therefore, some field observations used an artificial catchbasin loading from fire hydrants (Pisano, 1985). For this reason the present study compared four ICD's on a hydraulic model at 1:1 scale in the hydraulic laboratory at the University of Ottawa (Fig. 5). ICD performance was observed in a plexiglass catchbasin and flow measurements were made with a triangular weir.

The three ICD types commercially available and tested at the University of Ottawa are:

A. SCEPTER - an orifice type of ICD with a self-cleaning notch (Townsend, 1984).

B. CROMAC - an orifice type with a vcariable area slot ICD.

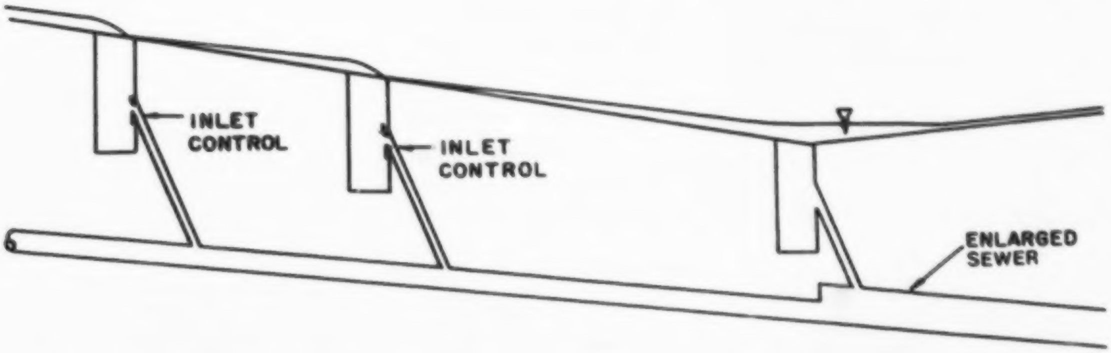


Figure 3a Inlet Control to reduce the Sewer Reconstruction Cost

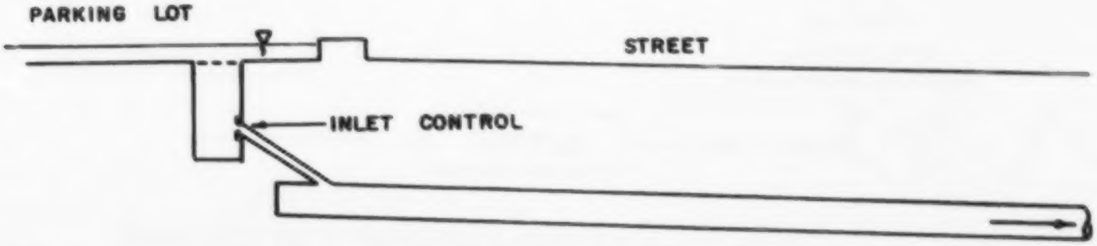


Figure 3b Parking Lot Inlet Control

C. HYDROVEX - a vortex ICD, representing an improved German version of HYDROBRAKE orifice.

The forth type called the HANGING TRAP is a self made ICD proposed in the Skokie studies. The schematics of different devices are shown in Fig. 6.

Head-Discharge curves were determined and found to be practically the same as those indicated by the manufacturers. The discharge coefficients are relatively the same for the first two devices, but are much small for HYDROVEX.

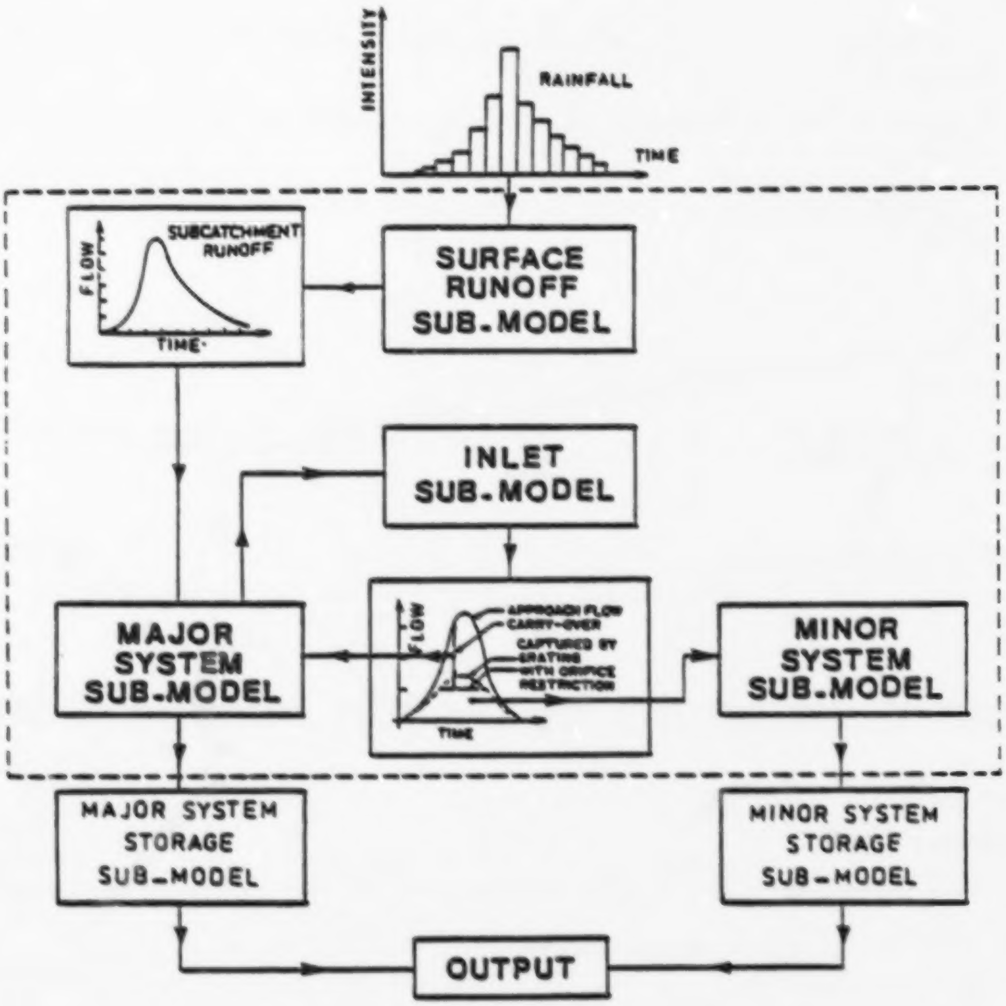
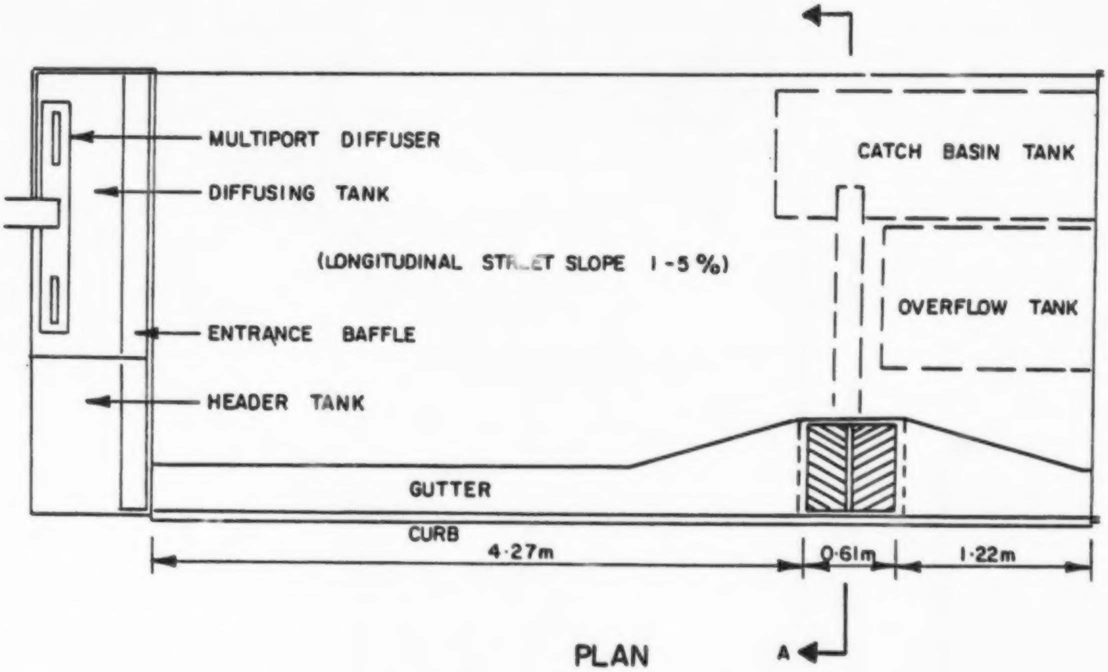

INTENSITY
RAINFALL
TIME
FLOW
SUBCATCHMENT RUNOFF
TIME
SURFACE RUNOFF SUB-MODEL
INLET SUB-MODEL
MAJOR SYSTEM SUB-MODEL
FLOW
APPROACH FLOW
CARRY-OVER
CAPTURED BY GRATING
WITH ORIFICE RESTRICTION
TIME
MINOR SYSTEM SUB-MODEL
MAJOR SYSTEM STORAGE SUB-MODEL
MINOR SYSTEM STORAGE SUB-MODEL
OUTPUT

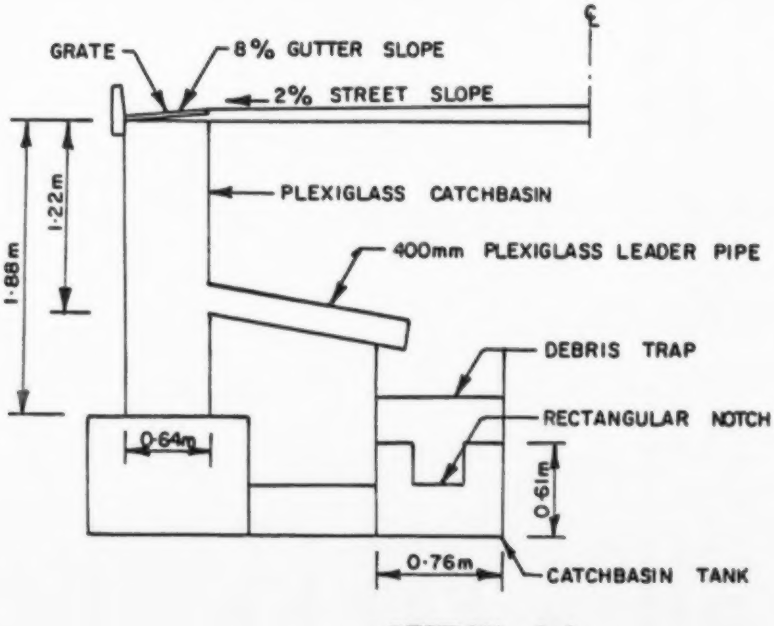
FIG. 4 : AN OVERVIEW OF THE UNIVERSITY OF OTTAWA STORMWATER MANAGEMENT MODEL (OTTSWMM)

TABLE 1 - EXAMPLES OF CASE STUDIES USING INLET CONTROL DEVICES

CASE STUDY	AREA HA.	LAND USE	AV.IMP. (%)	TOTAL NO. OF INLETS	INLET DENSITY (CB/ha)	%INLETS WITH RESTRICTORS	LEVEL OF INLET CONTROL (LPS)	SURCHARGE ELIMINATED	MAX.STREET FLOW DEPTH (cm)	PARK STORAGE USED	EXTERNAL AREAS (ha)	COMMENTS
A	465	RESID	31	MDP STUDY			5 YEAR CONTROL	AVOIDED	28	YES		- A detailed inlet control analysis was not conducted. ICD's used mainly to avoid surcharging the sewers during rare storms.
B	117	RESID	30	320	2.73	25	42	Limited & Acceptable		YES		-
C	10	RESID	25	32	3.2	75	42	Negligible	20	NO	15.4	
D	32	RESID	40	100	3.1	50	34 @ 28 lps 16 @ 42 lps	Limited & Acceptable	24	NO		-
E	44	RESID	35	166	3.7	51	28	Limited & Acceptable	26	YES		-
F	118	RESID	35			100	28	ELIMINATED	37	YES	90.8	- Severe inlet controls were required because of the status of the project at time of analysis
G	12	RESID	25	50	42	68	28	Limited & Acceptable	12	NO		-

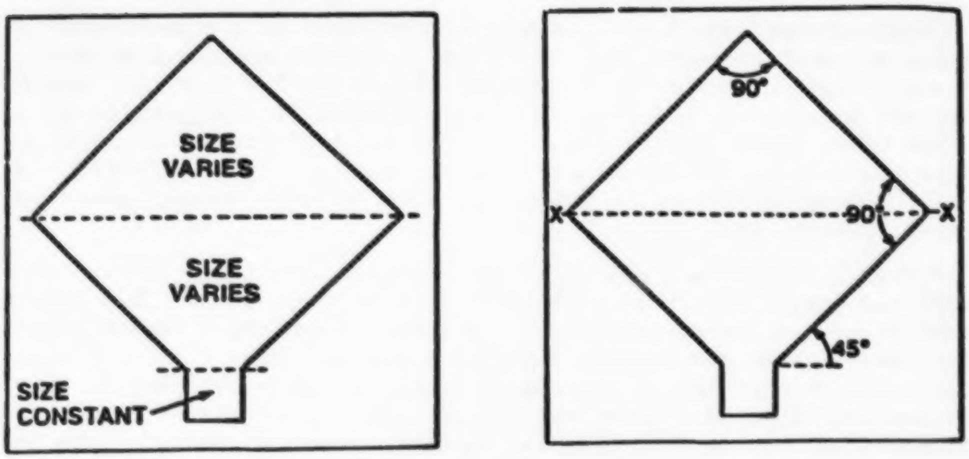

MULTIPORT DIFFUSER
DIFFUSING TANK
(LONGITUDINAL STREET SLOPE 1-5 %)
ENTRANCE BAFFLE
HEADER TANK
CATCH BASIN TANK
OVERFLOW TANK
GUTTER
CURB
4·27m
0·61m
1·22m
A

PLAN

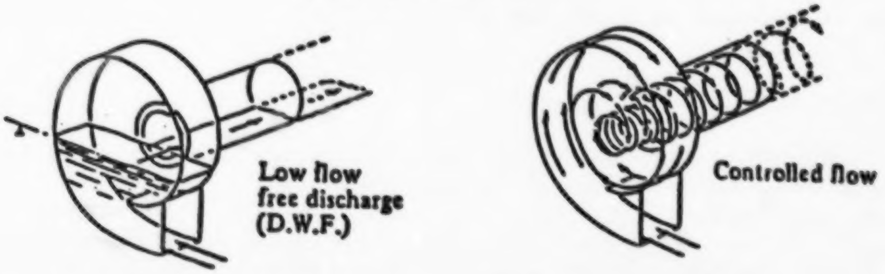

GRATE
8% GUTTER SLOPE
2% STREET SLOPE
1·88m
1·22m
PLEXIGLASS CATCHBASIN
400mm PLEXIGLASS LEADER PIPE
DEBRIS TRAP
RECTANGULAR NOTCH
0·64m
0·61m
0·76m
CATCHBASIN TANK

SECTION A·A

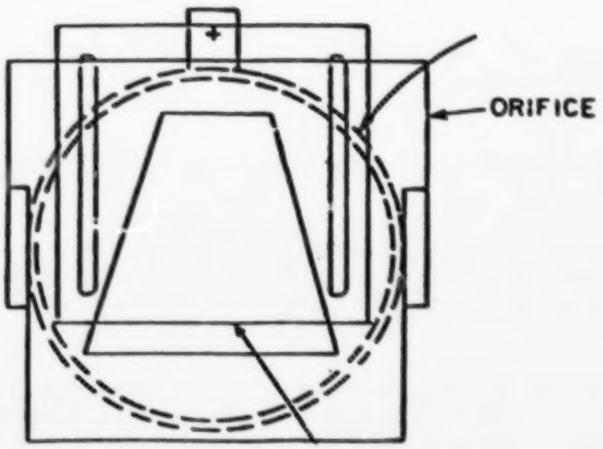
Figure 5 The Physical Model


SIZE
VARIES
SIZE
VARIES
SIZE
CONSTANT
90°
X
90°
X
45°

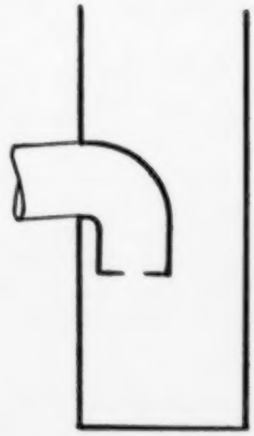
A) Schematic of the Scepter
Inlet Control Device


Low flow
free discharge
(D.W.F.)
Controlled flow

B) Schematic of the Hydrovex Unit in Operation



C) Schematic of the Cromac ICD



D) Hanging Trap ICD

Figure 6 Inlet Control Devices

Clogging experiments were in general conducted in a conservative mode as compared to natural occurences. The limit found for operation without permanent clogging was 20 lps for CROMAC and 14 lps for SCEPTER. HYDROVEX was tested and operated well down to 8.5 lps, although it may function adequately for even lower flows than in the test. For all ICD's it was found that certain combinations of leaves and branches loading may temporarily plug the orifice. Consequently, it is considered that periodic visual inspection and cleaning is necessary.

Hydraulic laboratory findings were compared with field experience in Markham and Scarborough, municipalities near Toronto, where ICD's have been successfully used for several years. A group of municipal engineers attended demonstration tests and presented useful comments. As a result of these discussions, another factor introduced in the selection process is the effect of protruding ICD's on current catchbasin cleaning practice. Based on these considerations, for typical Canadian separated sewer catchbasins the HANGING TRAP device was not recommended. The cost of HYDROVEX is at present higher than that of the simpler orifice type ICD's and it is therefore useful only if a low level of control is required. This, however, has other implications, which can be examined with OTTSWMM.

SELECTION OF AN INLET CONTROL DEVICE

As indicated above, the present practice in new Canadian subdivisions is to use a minimum control level of 28 lps. This limit can be reduced as low as 14 lps and orifice ICD's (SCEPTER) could still be used. For a lower level, the HYDROVEX is the only recommended ICD. Operations and cost implications were analyzed with the OTTSWMM model for five levels of control in a 42 ha typical subdivision, shown in Fig. 7. The OTTSWMM model sized conduits to avoid surcharge for the 100-year storm (Keifer and Chu distribution in Metro-Toronto).

Reducing the control level of ICD's gives pipe flow equivalent to a traditional design for a more frequent storm. As an example a 2-year storm, corresponds for Metro-Toronto conditions to a 20 lps ICD.

Results are summarized in Table 2. It was found for example that for a 20 lps ICD, park storages for overland flow would operate about twice per year instead of once every five years as for the 42 lps ICD currently used. The depth in the parks for this more frequent flooding would, however, be relatively small. The increase in maximum street flow depth at the gutters would be very small as compared to the present practice (1-4 cm for a 100-year storm).

Table 2 is an example of information available to a decision maker from the OTTSWMM model. It is found that if the level of control is lowered to 28 from 42 lps currently used in Ontario for ICD's in new subdivision, the savings may be significant. For example, in a typical residential area, inlet control at 20 lps per catchbasin can be achieved with the less expensive orifice type ICD's. The corresponding saving is $5,500/ha. Of course municipalities now using a 10-year or larger storms can achieve greater cost reductions.

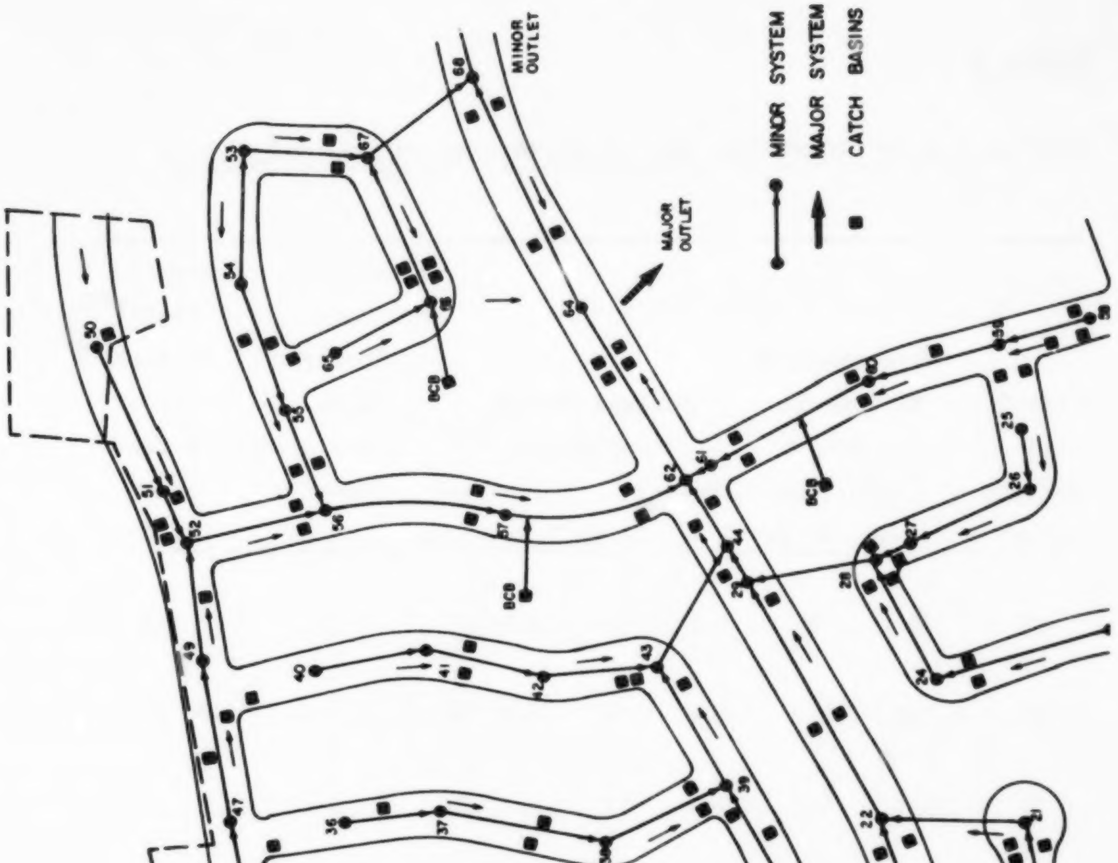

MINOR SYSTEM
MAJOR SYSTEM
CATCH BASINS
MINOR OUTLET
MAJOR OUTLET
BCB
BCB
BCB
BCB

Figure 7 Studied System

Table 2

COMPARISON OF ECONOMIC AND OPERATIONAL CHARACTERISTICS

Level of Control (lps)	Approximative Savings in $1,000/ha		Maximum Gutter Depth (cm)				Area of Park Storage % of Total Area	Average Depth in Park for a 5-Year Storm (cm)
	100-Year		5-Year		100-Year		100-year	
	7.3 ha	42 ha	7 ha	42 ha	7 ha	42 ha	42 ha	
42	0	0	7.4	14	13.7	27	1.1	18
28	2.9	3.2	8.4	16	15.0	30	1.4	17
20	4.6	5.5	9.6	18	15.7	31	1.8	18
14.0	4.7	6.6	10.2	20	16.2	33	2.2	20
8.5	6.4	8.6	10.9	22	16.5	34	2.7	23

The main limitation in achieving higher savings with inlet controls of HYDROVEX type, is the concern related to higher street flow depth during major storms and increased frequency of park flooding. These effects vary with contributing areas, imperviousness, slopes, etc. and can be assessed using detailed OTTSWMM computations for the specific conditions of each project.

The sewer network or minor system can follow either a dendritic or looped pattern. When a new system is being designed, the pipes should follow a dendritic pattern. More sophisticated analysis of looped or surcharged pipes is possible with the EXTRAN (Extended Transport) submodel. The pipe slopes control the flow direction in the minor system. Water enters the minor system through storm inlets which are connected to manholes at sewer junctions. At each junction the flow from upstream pipes is added to the street inlet flow giving the total flow to be routed to the next segment. Pipes are sized for the peak flow at each junction.

MODEL OPERATION

The model is composed of four main submodels, a surface runoff submodel, inlet submodel, minor system submodel and the major system submodel.

The input data consisting of subareas, street segments, pipe segments, and storages are read and connectivity matrices set up. The input data order is shown in Figure C.1. Computations are done in a number of steps. First the runoff for each subarea is computed using the Runoff Block routine borrowed from EPA-SWM Model. This is followed by the major system routing. Starting at an upstream major system segment subarea runoff is routed down the street segment with any upsteam carryover flow. In conjunction with the major system routing, inlet flows are determined. Any flow captured by the inlets are stored for minor system analysis. Excess flow not caputred by the inlets form carryover flow to be routed down the following major system segment. The street segment routing and inlet capture are continued until all of the street segments have been considered.

The computations for the minor system components are performed next. The user selected input determines how the pipe system is analyzed. Design of a new system starts with the most upstream sewer segment and proceeds downstream. The pipe flow is the total of inlet capture flow and the upstream pipe flow. Design or resizing of individual pipes is performed with the MINOR submodel. When the pipes are sized, free surface and a dendritic sewer system is assumed. Pipes are selected based on the input slopes, pipe roughness and computed peak flow, using the Manning equation. The model selects the smallest commercial sewer size that will maintain free surface flow.

If the pipe surcharge analysis option is selected the inlet flows are saved in a separate file, to be used by EXTRAN subroutine. Analysis with EXTRAN can be conducted in looped pipe systems and in systems where pipes have insufficient capacity for free surface flow. Some small surcharging may be desirable during major storm events, to prevent resizing of surcharged pipes. Surcharge levels must be kept below foundations, to prevent basement flooding damage. The EXTRAN model must be used to determine if the surcharge levels are acceptable.

ARE MODELS SAVING MONEY?

The change of a well entrenched methodology requires not only studies but also pilot projects and innovative municipal engineers. The first ICD-dual drainage project in Markham, Ontario was proposed in 1978. At present the system is used in other Metro-Toronto municipalities (Vaughan, Scarborough) and is recommended by criteria in other smaller cities (Oakville, Barrie). It was also applied in various projects in Nepean, Cumberland, etc. Some initial applications were implemented after flooding experiences.

The proposed reduction of pipe sizes, is a next step and may require a dialogue with decision makers and developers. The use of OTTSWMM for new subdivisions showns that modelling can be used for a more economic drainage design.

A previous IMPSWM study (Wisner, Cheung 1982) developed an improved rational method in which the runoff coefficient is a function of imperviousness and the inlet time varies with rainfall intensity. This method, available for microcomputers under the name IMPRAM gives the peak flows in close agreement with SWMM model (Fig. 8).

It follows that for a conventional design the use of a more sophisticated model does not offer an advantage as compared to a slightly improved traditional computation. In the case however, of the sophisticated dual drainage-inlet control-park storage design, modelling is justified by significant savings and evaluation of a more realistic level of protection.

Experience shows that the OTTSWMM model is very simple to use, even if consultants are not familiar with the SWMM model. Huitt-Zollars, a consulting firm in Dallas, used OTTSWMM without previous SWMM experience on a relatively complex relief sewer study for a large downtown area in Dallas (Umble 1986). The study found that the role of gratings even without ICD's is very important. OTTSWMM users have support through the IMPSWM (Implementation of Storm Water Management) program at the University of Ottawa which operates a hot-line (613-564-3911 or 613-564-7022).

The main difficulties are of a different type:

- acceptance of the reality of overland flow by decision makers.
- integration of shallow storage parks for overland flows at strategic locations by planners.
- convincing consultants for developers to pay more attention to street grading and avoidance of low points.

If a minimum $2,000/acre saving is not an adequate incentive to overcome these difficulties, one should also consider the considerable damages caused by basement flooding in areas where foundation drainage is connected with storm sewers (Wisner, Hawdur, 1983).

ACKNOWLEDGEMENTS

Mr. D. Keliar, Director of Engineering, Town of Markham, Mr. Jean Claude Pigeon, Drainage-Director in Laval, Quebec and other engineers in Canada made a significant contribution to the implementation of OTTSWMM based applications of ICD's and dual drainage. This was seminal in generating further applications and particularly this study. The Ontario Ministry of Environment included the ICD-dual drainage concepts in its recent criteria and provided assistance for OTTSWMM.

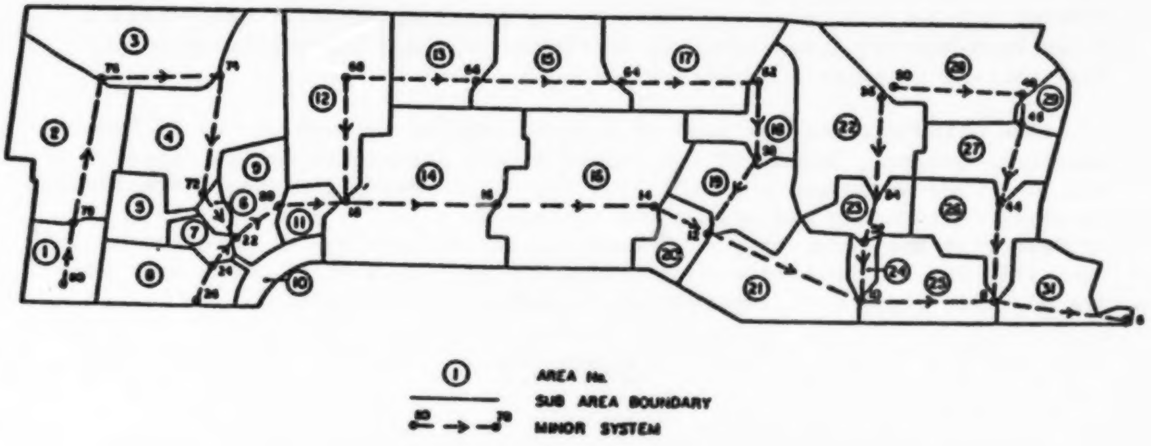

①
AREA No.
SUB AREA BOUNDARY
80
70
MINOR SYSTEM

Figure 8 Comparison of Peak Flows done with SWMM and IMPRAM* for the example area (ti = 10 min.)

Cumulative area (Ha)	SWMM (cms)	IMPRAM (cms)
3.27	0.32	0.3
4.52	0.44	0.4
5.90	0.55	0.5
8.94	0.83	0.7
10.52	1.00	0.8
12.42	1.19	0.9
13.96	1.34	1.1
15.72	1.34	1.5
18.37	1.78	2.0

* IMPRAM (Improved Rational Method) by IMPSWM available for microcomputers.

Mr. John Meunier, distributor of HYDROVEX, Mr. Bob Crosmas, developer of the CROMAC device attended the experiments and provided ICD devices. The SCEPTER ICD was also made available by the Manufacturerers. Drs. Stuart Welsh from Donahue Associates and W. Pisano provided documentation on ICD applications on existing sewer systems.

The OTTSWMM model was used in a microcomputer version of the IMPSWM mainframe program, developed for Andrew Brodie Associates by Mr. Kassem. Review of the study and comments from Mr. Martin Hawdur, P. Eng., President, Novatech Engineering and advice and discussion with Dr. Ron Townsend, Professor at the University of Ottawa, on the methodology of experiments and data from Mr. Andrew Brodie on his numerous applications of ICD's were very useful. All these contributions are gratefully acknowledged.

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APPENDIX

DUAL DRAINGE COMPUTER MODEL - OTTSWMM

Most urban storm drainge models assume that all the catchment runoff is transferred directly into the minor system. They have been developed for designing or analyzing systems with low return period rainfall events. However, if the dual drainage conept is to be employed, drainage systems must be designed for events with a high return period. In this case, all the storm water runoff is not captured by catchbasins and transferred into the pipe system. A portion of the street flow is captured and transferred to the pipe system while the remaining carry over flow is transported by the streets.

The OTTSWMM model was designed specifically for analyzing dual drainage systems. The program has the capability of determining the surface flow, the hydraulic capture by catchbasin inlets and the pipe flow. It can be used in four modes:

i) to determine pipe sizes for free surface flow;
ii) to analyze an existing system or proposed design and resize pipes to maintain free surface flow;
iii) to determine the level of inlet restriction to maintain free surface flow in pipes;
iv) to conduct a pipe surcharge analysis.

Whatever mode of operation is being used, the basic assumptions of the model remain the same. The model conducts an analysis of two interconnected systems, the surface or major system and the pipe or minor system. Since the computations are done on two levels, the surface and sewer network flows do not necessarily have to be in the same direction.

The major system is formed by the street network and must follow a dendritic pattern converging to a downstream outlet. The major system should be continuous, no water ponding is allowed except at storage locations. In new subdivision, streets can be designed so that low points are avoided. Existing development often have low points. In recognizing this the model permits two types of inlets. Normal inlets are those where flow partly enters the minor system and is partly passed down the major system. Storage inlets are located at low points, all of the water enters the minor system at these inlets.

The sewer network or minor system can follow either a dendritic or looped pattern. When a new system is being designed, the pipes should follow a dendritic pattern. More sophisticated analysis of looped or surcharged pipes is possible with the EXTRAN (Extended Transport) submodel. The pipe slopes control the flow direction in the minor system. Water enters the minor system through storm inlets which are connected to manholes at sewer junctions. At each junction the flow from upstream pipes is added to the street inlet flow giving the total flow to be routed to the next segment. Pipes are sized for the peak flow at each junction.

MODEL OPERATION

The model is composed of four main submodels, a surface runoff submodel, inlet submodel, minor system submodel and the major system submodel.

The input data consisting of subareas, street segments, pipe segments, and storages are read and connectivity matrices set up. The input data order is shown in Figure C.1. Computations are done in a number of steps. First the runoff for each subarea is computed using the Runoff Block routine borrowed from EPA-SWM Model. This is followed by the major system routing. Starting at an upstream major system segment subarea runoff is routed down the street segment with any upsteam carryover flow. In conjunction with the major system routing, inlet flows are determined. Any flow captured by the inlets are stored for minor system analysis. Excess flow not caputred by the inlets form carryover flow to be routed down the following major system segment. The street segment routing and inlet capture are continued until all of the street segments have been considered.

The computations for the minor system components are performed next. The user selected input determines how the pipe system is analyzed. Design of a new system starts with the most upstream sewer segment and proceeds downstream. The pipe flow is the total of inlet capture flow and the upstream pipe flow. Design or resizing of individual pipes is performed with the MINOR submodel. When the pipes are sized, free surface and a dendritic sewer system is assumed. Pipes are selected based on the input slopes, pipe roughness and computed peak flow, using the Manning equation. The model selects the smallest commercial sewer size that will maintain free surface flow.

If the pipe surcharge analysis option is selected the inlet flows are saved in a separate file, to be used by EXTRAN subroutine. Analysis with EXTRAN can be conducted in looped pipe systems and in systems where pipes have insufficient capacity for free surface flow. Some small surcharging may be desirable during major storm events, to prevent resizing of surcharged pipes. Surcharge levels must be kept below foundations, to prevent basement flooding damage. The EXTRAN model must be used to determine if the surcharge levels are acceptable.

Storage can be provided to control either minor or major system runoff. The minor system storage may be provided by a superpipe, or other in system detention facility. For the major system, street flow is typically directed

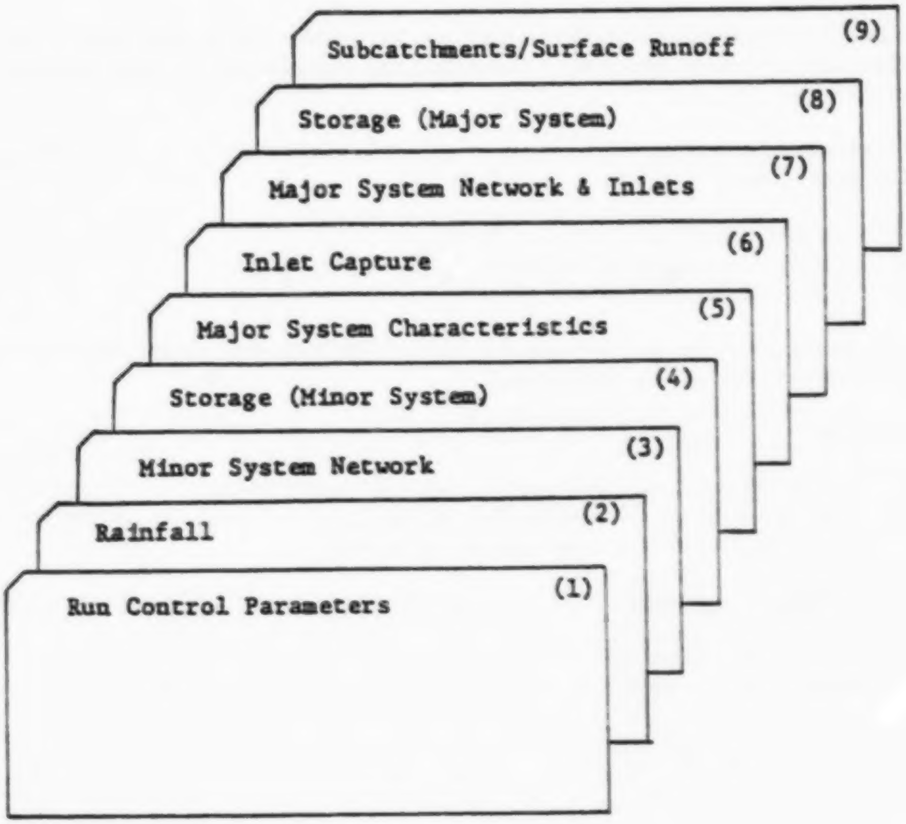
Subcatchments/Surface Runoff (9)
Storage (Major System) (8)
Major System Network & Inlets (7)
Inlet Capture (6)
Major System Characteristics (5)
Storage (Minor System) (4)
Minor System Network (3)
Rainfall (2)
Run Control Parameters (1)

Fig. C1 Main Card Groups For Input Data to OTTSWMM

to a surface storage facility such as a depressed park or detention pond. Minor system storage is provided to obtain economies in pipe sizing or to ensure that peak flows do not exceed predevelopments levels. Major system storage is necessary to intercept street flow, and release it at a controlled rate into the minor system or receiving stream. The designer inputs the stage-discharge curve to provide the desired control, the computer output gives the required storage volumes. If a storage volume was input and it is exceeded the overflow volume is given in the output.

The basic computer output of OTTSWMM provides the following information:

1. a print-out of the input data as well as a summary statistics of the watershed: number of subareas, sewers, storage units, total drainage area, etc., density of inlets (number of inlets per unit area), average distance between inlets, etc.;

2. required sizes of sewers for free surface flow conditions;

3. inlet control requirements, that is locations of inlets which may need flow constricting devices, and limiting capacities if the latter is not specified;

4. detailed simulation results for specified elements, in printed and plotted forms:

   a) time history of surface runoff;
   b) time histroy of major system flows and depths;
   c) time history of sewer flows;

5. a summary of simulation results including maximum flows and depths at various locations as requested.

6. Storages:

   a) required storage volume for major system flow;
   b) required storage volume for minor system flow, or volume of overflow if a given storage volume is exceeded.

7. with EXTRAN for sewer flow routing the following information is obtained:

   a) printout summary for water depth at junctions;
   b) printout summary for conduits showing design flows.

The work described in this paper was not funded by the U.S. Environmental Protection Agency and therefore does not necessarily reflect the views of the Agency and no official endorsement whould be inferred.

USE OF CONTINUOUS SWMM FOR SELECTION OF HISTORIC RAINFALL DESIGN EVENTS IN TALLAHASSEE

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ABSTRACT

Conventional design methods often utilize a "design storm" synthesized from a total depth taken from an intensity-duration-frequency curve for an arbitrary duration, coupled with an assumed temporal distribution using any of several available shapes for the hyetograph. The synthetic design storm is then used as input to a rainfall-runoff model. Unfortunately, the true return period of runoff parameters such as peak flow or volume is unknown since antecedent conditions must be arbitrarily assumed when using such a method. In addition, there is seldom a firm basis for the choice of the storm duration or temporal distribution. Nonetheless, the use of this method is very common.

As an alternative, a model may be calibrated and verified for the drainage basin and then used in a continuous simulation for as many years as there are available rainfall data. A frequency analysis of the predicted runoff events may then be used to select historic rainfall events for use in design based on desired return periods on the parameter of interest, such as peak flow, runoff volume, flood stage, pollutant load, or pollutant concentration. The historic rainfall events so identified may then be simulated in more detail for design purposes.

The results of such an analysis for the Megginnis Arm Catchment in Tallahassee, Florida are described in the paper. SWMM simulations with historic and synthetic design storms are compared with respect to a frequency analysis of peak flows. Contrary to what is often assumed, it is found that the synthetic design storms are not necessarily conservative (i.e., they do not necessarily produce higher peak flows) over the range of return periods considered (up to 25 years).

INTRODUCTION

ANALYSIS OF URBAN STORMWATER

Urban drainage studies may be conducted for several purposes, including flood-control and water quality control. "Flood control" may simply mean conventional drainage of urban streets or be more comprehensive in nature, e.g., for a whole basin. The usual design criterion is peak flow, although stage (or hydraulic grade line) is likely more important. Runoff volumes are also of interest when storage is used as a control measure. Quality control may be applied to limit pollutant loads to downstream receiving waters and may or may not necessitate a detailed simulation of a pollutograph (concentration versus time) during a storm. In most cases, receiving waters are sensitive only to the total load and not detailed variations within a storm (Driscoll, 1979; Hydroscience, 1979); hence, in most cases, only total loads need to be predicted.

For both quantity and quality control, design conditions must be specified. In some instances, the design engineer has little choice. For instance, in Florida, the Department of Environmental Regulation (DER) specifies in part that stormwater quality criteria can be met by providing for the retention of the runoff resulting from the first one inch or rainfall or of the first half-inch of runoff, for projects of less than 100 ac. This leaves few design options to the engineer, although it is easy to administer.

In the area of drainage design, another common guideline promulgated by agencies is to specify the use of a synthetic design storm. This is usually constructed by obtaining the depth for a given frequency and duration (e.g., 25-yr, 24-hr) from an intensity-duration-frequency (IDF) curve. The depth is then distributed in time by using an assumed temporal distribution of the storm, e.g., the Soil Conservation Service (SCS) Type II distribution for the hyetograph. What is usually unknown and ignored in such decisions is the true return period of the design. What level of protection has really been afforded by this design? If the desired level of protection is, say, 25 years, based on rainfall statistics, are the statistics (frequencies) of peak flows or volumes consistent with this value? It is easy to comprehend that the return period of a rainfall event is unlikely to be the same as the return period of the flow peak or volume caused by the event because of variable antecedent conditions and the inherent nonlinearity of the catchment. Is the resulting design conservative (over designed) and therefore uneconomic, or the reverse? It is difficult to say *a priori*, although comparisons have been made (Marsalek, 1978; Arnell, 1978,1982). However, most conventional wisdom assumes that synthetic design storms are conservative, a result that will be shown not always to be true. For quality, Geiger (1984) presents convincing combined sewer overflow data showing that return periods of event mean concentrations can be quite different from the rainfall and runoff that caused them.

The choice of rainfall input for models has been widely studied, e.g., by Adams and Howard (1985), Arnell (1978, 1982), Harremoes (1983), Huber et al. (1981), James and Robinson (1982), Marsalek (1978), McPherson (1978), Patry and McPherson (1979), Wenzel and Voorhees (1981). Adams and Howard (1985) make a particularly strong argument against the use of design storms, synthetic or historic, and recommend continuous simulation or derived frequency distributions.

This paper will illustrate the use of continuous simulation for selection of historic design rainfall events for use in detailed design simulations. In this method, a model is first calibrated and verified for the catchment. A continuous simulation is then performed using as long a record as possible of hourly or shorter increment rainfall data (about 25 years of computerized hourly values are typically available from the U.S. National Weather Service). A frequency analysis is then performed on the parameters of interest, such as peak flow, runoff volume, or quality loads and concentrations, from which historic rainfall events that produce a runoff event of the desired return period are identified. Finally, the storms may be input again to the model in a more detailed design simulation. By this means, questions of antecedent conditions, storm duration, and storm shape are avoided, and a better idea of the (unknown) "true" return period for the design is obtained.

Why not use measured flows for this purpose? Although this indeed might resolve the question of the true design frequency, it is seldom possible in urban areas because of lack of a gage altogether, a short or incomplete record, or changing land use during the time of the gaging. Simulation offers a less exact option to bypass these difficulties.

OBJECTIVES

Ultimately, it is desirable to determine the relative effect of using various methods for drainage design on the design condition. This paper will partially address this question for peak flows using one catchment as a case study. Eventually it may be even more important to determine the economic impact of alternative design methodologies, both on the cost of the project and on the cost of the engineering design. In other words, if the use of more sophisticated and complex methods only results in a marginally different answer from that obtained by conventional means then it may be constructed for almost the same cost either way, but be much cheaper to design using the conventional (simpler) methods. This latter question of costs is currently under study at the University of Florida and will not be addressed in this paper.

The immediate objectives of this paper are thus to:

1. Determine design storms from historic rainfall series using continuous simulation, and

2. Compare peak flow results obtained by simulation with historic design storms, synthetic design storms and other methods.

METHODS

The options will be compared through a case study using the Megginnis Arm Catchment in Tallahassee, Florida. The steps are enumerated below.

1. Calibrate and verify the SWMM model on the catchment.

2. Perform a continuous simulation using the 22 year historic record of hourly rainfalls (1958-79).

3. Perform a frequency analysis on predicted peak flows and runoff volumes.

4. Select historic rainfall events that give peaks or volumes of desired return periods.

5. Run these historic storms through the SWMM model again using a more detailed simulation and compare with the results using synthetic design storms and other procedures.

6. Show the final comparisons on a plot of peak flow versus return period for the different methods.

SWMM SIMULATIONS

WHY SWMM?

The EPA Storm Water Management Model (Huber et al., 1981; Roesner et al., 1981) was chosen because of its many applications to urban areas (Huber et al., 1985) and because it may conveniently be used for both continuous and single event simulation. Alternative choices include STORM (Roesner et al., 1974; HEC, 1977) and HSPF (Johanson et al., 1980). However, STORM is not well suited for single event simulation and generally has only simple hydrologic and water quality routines (which do not detract from its usefulness as a planning tool). HSPF might be a viable alternative, but it is less well suited to urban areas than is SWMM.

THE CATCHMENT

The 2230 acre Megginnis Arm Catchment in Tallahassee, Florida (Figure 1) discharges to Lake Jackson north of the city and has experienced both quantity and quality problems in recent years due to increased urbanization. A runoff data base exists since 1973, with rainfall-runoff data available since 1979 (both collected by the USGS). It is the site of a multi-million dollar experimental water quality control facility consisting of a detention basin and artificial marsh, and has been included by the USGS in their urban runoff studies (Franklin and Losey, 1984). A summary of reports available for the catchment is given by Esry and Bowman (1984). Preliminary applications of the Runoff Block of SWMM have been reported by Huber et al. (1986) for purposes of predicting 5-year peak flows. The more comprehensive results of the continuous and single event simulations will be illustrated here and compared with alternative engineering analyses of the basin for prediction of peak flows. The various methodologies will be compared over a range of return periods, although there will be considerable uncertainty in the peak flow estimates at the end of the 22 year rainfall record. The 5-year results are more reliable since they fall in the middle of the range of return periods and will be singled out for special consideration of the historic storms involved.

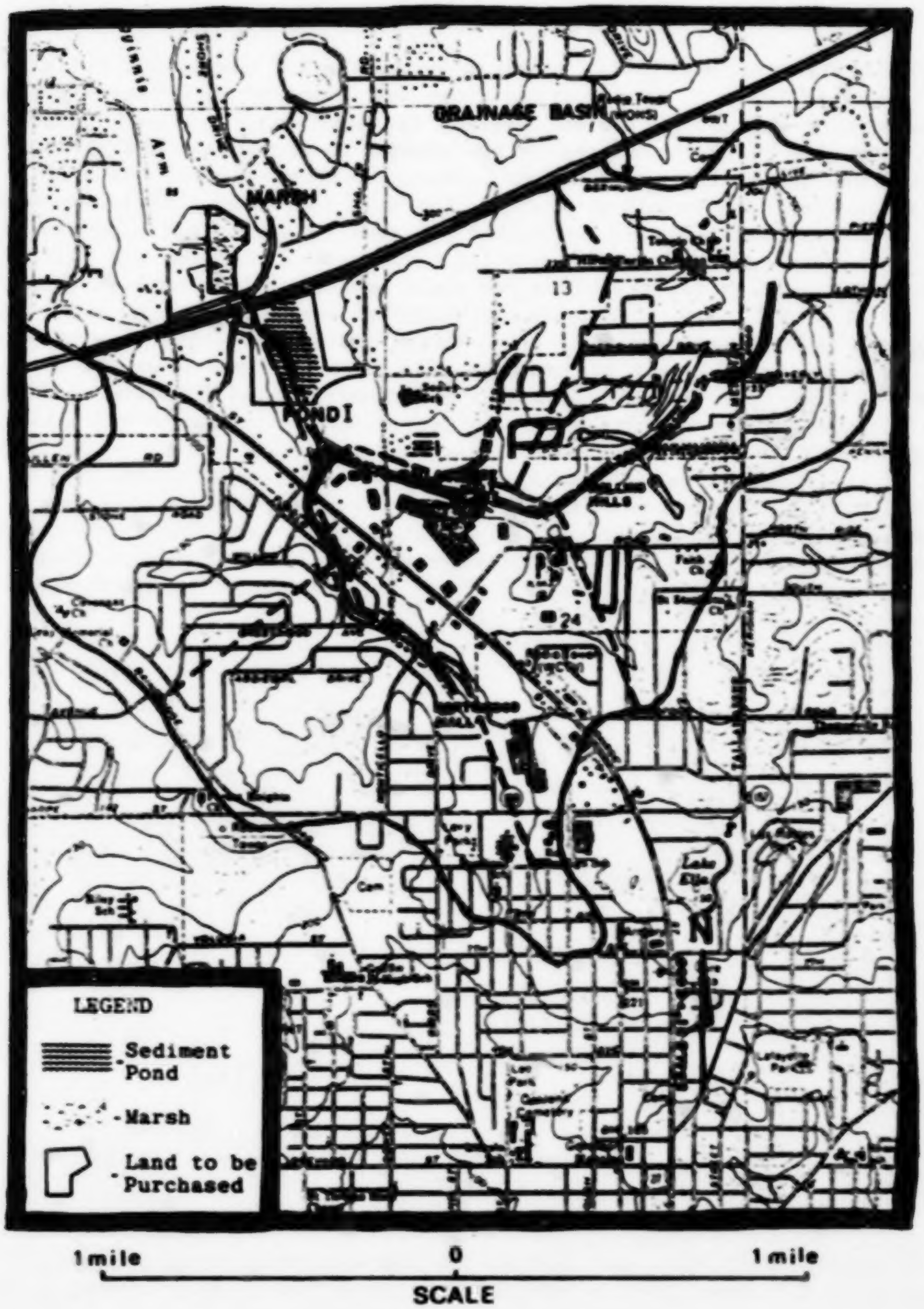

DRAINAGE BASIN
MARSH
POND I
13
24
N
LEGEND
Sediment Pond
Marsh
Land to be Purchased
1 mile
0
1 mile
SCALE

Figure 1. Megginnis Arm Catchment in Tallahassee, Florida. Rainfall and flow measurements are made at the entrance to Pond I.

SWMM CALIBRATION

Calibration of SWMM first required estimates for the Runoff Block parameters listed in Table 1. The average slope was found by selecting eight points at the edge of the catchment, calculating the path length of each point to the inlet, dividing the path lengths by the change in elevation, and taking a weighted average of the eight slopes. Percent imperviousness was obtained from an earlier USGS modeling study by Franklin and Losey (1984). Green-Ampt infiltration parameters were estimated by identifying the soils in the catchment (primarily sandy) from a county soil survey map, finding the hydraulic conductivity and capillary suction for each soil from data published by Carlisle et al. (1981), and then taking a weighted average over the soil types. Manning's n values were selected from charts based on average type of ground cover. Average monthly pan evaporation data were obtained from National Weather Service (NWS) values published by Farnsworth and Thompson (1982), from which actual evapotranspiration (ET) estimates were calculated by multiplying by a pan coefficient of 0.7. Final parameter estimates are shown in Table 1. The overall catchment was schematized using only one subcatchment and no channel routing in order to maintain a reasonable computation time for the continuous simulation. Later, a five subcatchment schematization was used, but the there was only a minimal improvement in the predictions. Hence, detailed simulation results are reported only for the single subcatchment schematization.

TABLE 1. RUNOFF BLOCK PARAMETERS FOR THE MEGGINNIS ARM CATCHMENT

Parameter	Value
Area	2230 ac (903 ha)
Width	6000 ft (1830 m)
Percent Imperviousness	28.3
Slope	0.0216
Manning's Roughness	
Impervious	0.015
Pervious	0.35
Depression Storage	
Impervious	0.02 in (0.5 mm)
Pervious	0.50 in (13 mm)
Green-Ampt Parameters	
Suction	18.13 in (461 mm)
Hydraulic Conductivity	5.76 in/hr (146 mm/hr)
Initial Moisture Deficit	0.15

Rainfall-runoff data for calibration and verification were obtained from available USGS records for the catchment. Ten of the largest storms were selected from the period 1979-81 and randomly divided into two sets of five storms each, one for calibration and the other for verification. The larger storms were chosen since the ultimate use of the modeling was to be drainage design, for which calibration for large storms is preferable. The model was calibrated for the five storms simulataneously, that is, while maintaining the same parameter values for each (Maalel and Huber, 1984). Calibration of runoff volumes was sufficient using the assumed value of catchment imperviousness (Figure 2); thus, calibration for peak flows was achieved only by varying the subcatchment width (a parameter in SWMM that is equivalent to making changes in the slope or roughness). The results for peak flows are shown in Figure 3.

Verification was accomplished by running five different storms using the same parameters that were used in the final calibration runs. Results of the verification runs were comparable to the calibration runs and are also shown in Figures 2 and 3. Individual storms exhibit varying goodness of fit as shown in Figures 4-7. Figure 4 illustrates the best of the ten fits and Figure 5 the worst, while Figures 6 and 7 illustrate typical intermediate results. It should be emphasized that antecedent condtions were not altered for individual storms which would have aided in the fits. (Continuous simulation eliminates this problem.) However, a robust calibration was desired, which would produce a good fit, on the average, for many storms (Maalel and Huber, 1984). Hence, the agreement between measured and predicted volumes and peaks shown in Figures 2 and 3 was considered adequate for this study.

FREQUENCY ANALYSIS

Upon completion of the calibration and verification exercises, a continuous run of 21.6 years (259 months, June 1958 to December 1979) was made using hourly rainfall data from the Tallahassee Airport NWS rain gage, located approximately 7 miles southwest of the study area. Statistical analysis of the predicted flows was performed using the Statistics Block of SWMM. The time series of hourly runoff values was separated into 1485 independent storm events by varying the minimum interevent time, MIT, until the coefficient of variation of interevent times equalled 1.0, yielding a value of MIT = 19 hours. This method of delineating independent events (Hydroscience, 1979; Restrepo-Posada and Eagelson, 1980) is based on the fact that the exponential distribution is often fit to interevent times, and it has a coefficient of variation (standard deviation divided by the mean) equal to 1.0.

The SWMM Statistics Block performs a frequency analysis on any or all of the following parameters: runoff volume, average flow, peak flow, event duration, and interevent duration. (If pollutants were being simulated, frequency analyses could also be performed on: total load, average load, peak load, flow weighted average concentration, peak concentration.) Storm events are sorted and ranked by magnitude for each parameter of interest, and assigned an empirical return period in months according the the Weibull formula ($T = (n+1)/m$, where n is the total number of months and m is the rank of an event). The largest magnitude event for this 259-month simulation was thus assigned a return period of 260 months. Hence, for this simulation, a five year event is bracketed by return periods of 52 and 65 months (fifth and fourth largest events, respectively), both of which were selected as "design events."

On this basis, the historic storms producing the nine highest peak flows

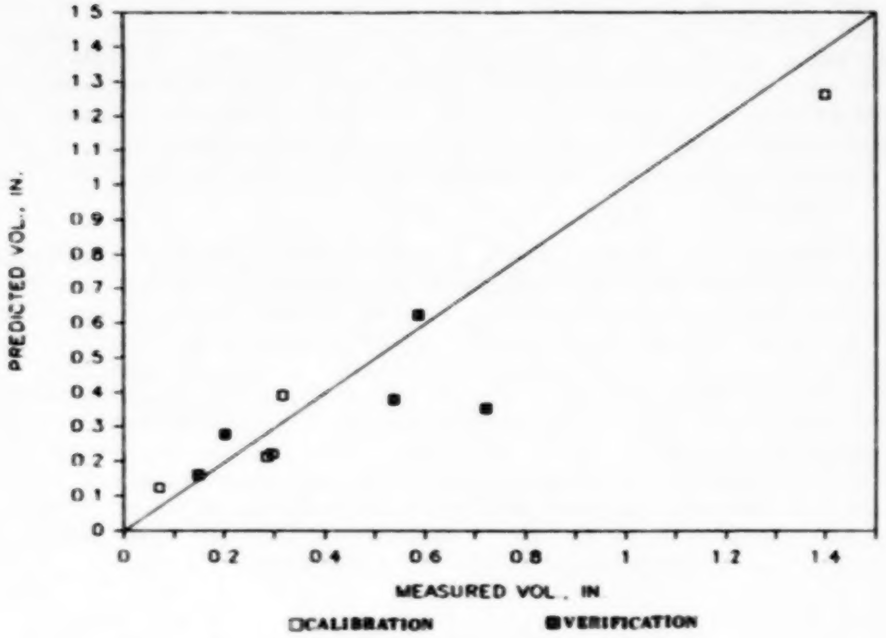


Figure 2. Goodness of fit of runoff volumes.

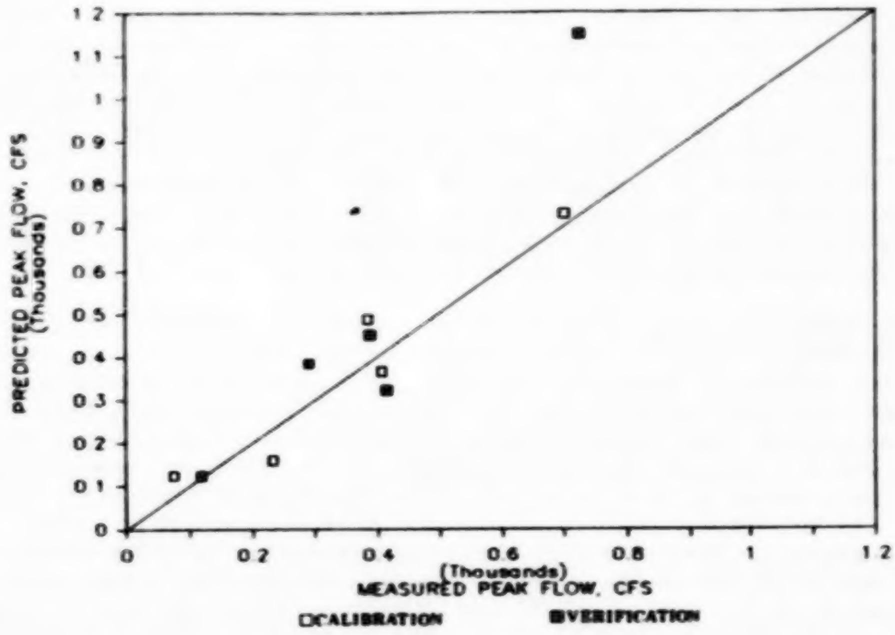
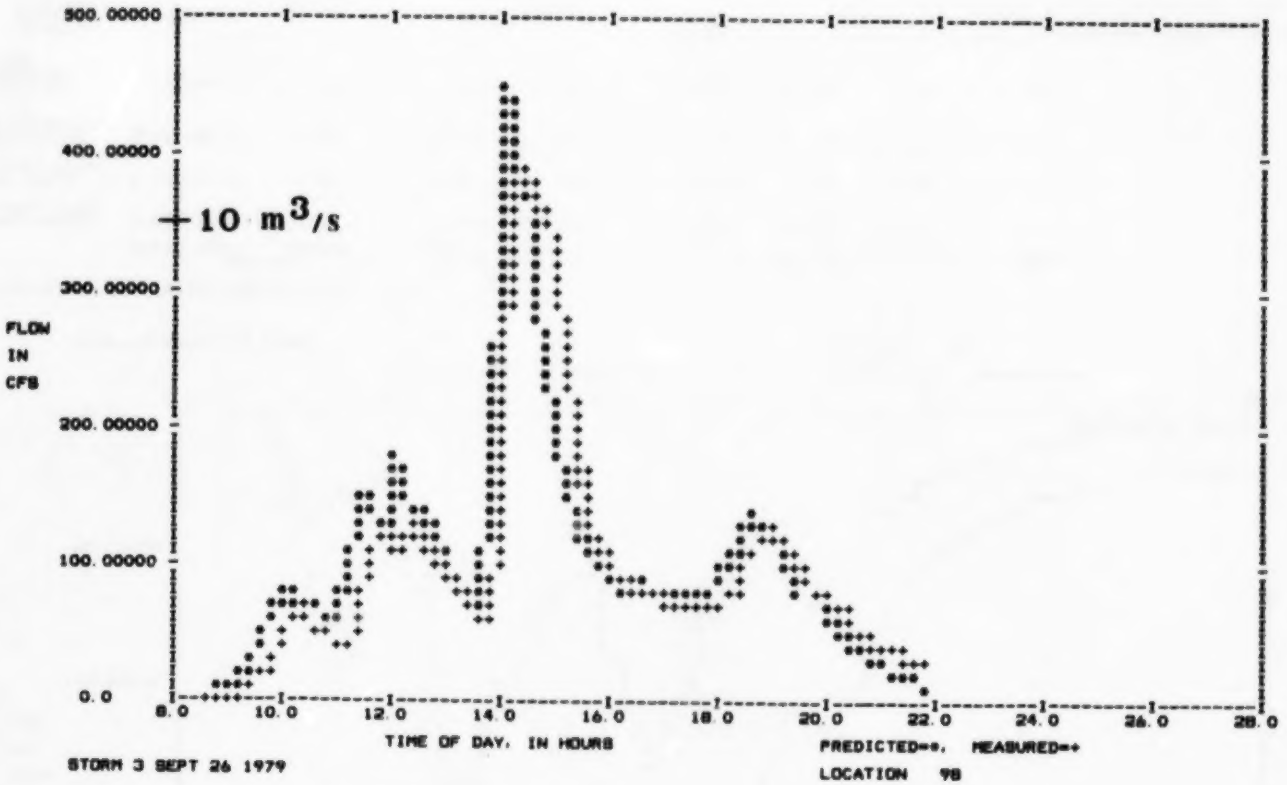


Figure 3. Goodness of fit of runoff peaks.



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HYDROGRAPH STATISTICS FOR LOCATION  98

                     VOLUME                PEAK FLOW                   DURATION                 NO.
              CUBIC FEET    INCHES      TIME,HR  FLOW,CFS     START,HR  END,HR   LENGTH,HR    POINTS
PREDICTED,    0.50472E+07    0.624      14.083   451.430       8.500    21.833   13.333        161
TOTAL TIME

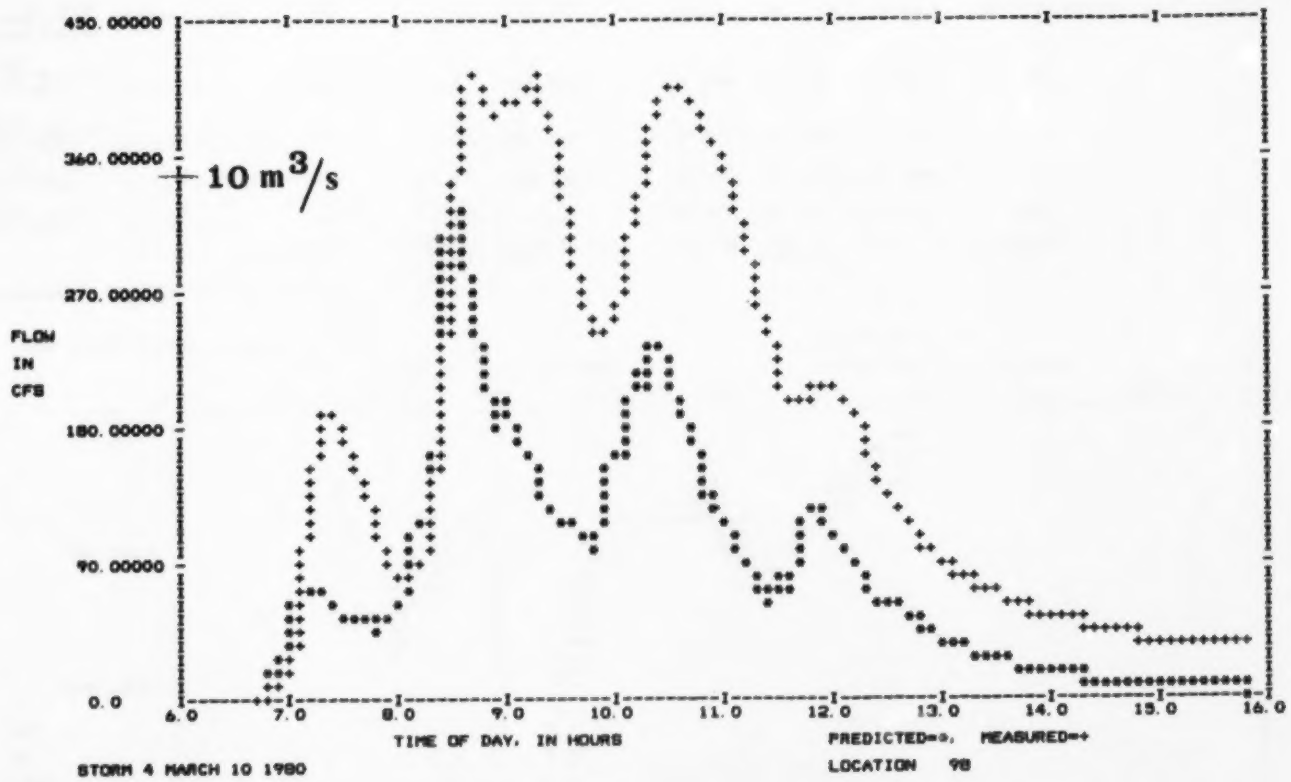
MEASURED,     0.47463E+07    0.386      14.417   387.000       8.500    21.750   13.250        160
TOTAL TIME

PREDICTED,    0.50429E+07    0.623      14.083   451.430       8.500    21.750   13.250        160
OVERLAPPING
TIME

MEASURED,     0.47463E+07    0.386      14.417   387.000       8.500    21.750   13.250        160
OVERLAPPING
TIME

DIFFERENCES,
  ABSOLUTE   -0.29647E+06   -0.037       0.333   -64.430
  % OF MEAS                 -6.246               -16.649
```

Figure 4. Predicted and measured hydrograph, September 26, 1979 (verification run).



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HYDROGRAPH STATISTICS FOR LOCATION  98
                      VOLUME              PEAK FLOW                 DURATION                 NO.
               CUBIC FEET    INCHES     TIME,HR  FLOW,CFS     START,HR  END,HR  LENGTH,HR   POINTS
PREDICTED,     0.28463E+07    0.352      8.583  321.659       6.667   15.833    9.167      111
TOTAL TIME
MEASURED,      0.58339E+07    0.721      8.667  414.000       6.667   15.750    9.083      110
TOTAL TIME
PREDICTED,     0.28450E+07    0.351      8.583  321.659       6.667   15.750    9.083      110
OVERLAPPING
TIME
MEASURED,      0.58339E+07    0.721      8.667  414.000       6.667   15.750    9.083      110
OVERLAPPING
TIME
DIFFERENCES,
  ABSOLUTE     0.29889E+07    0.369      0.083   92.341
  % OF MEAS                  51.233              22.305
```
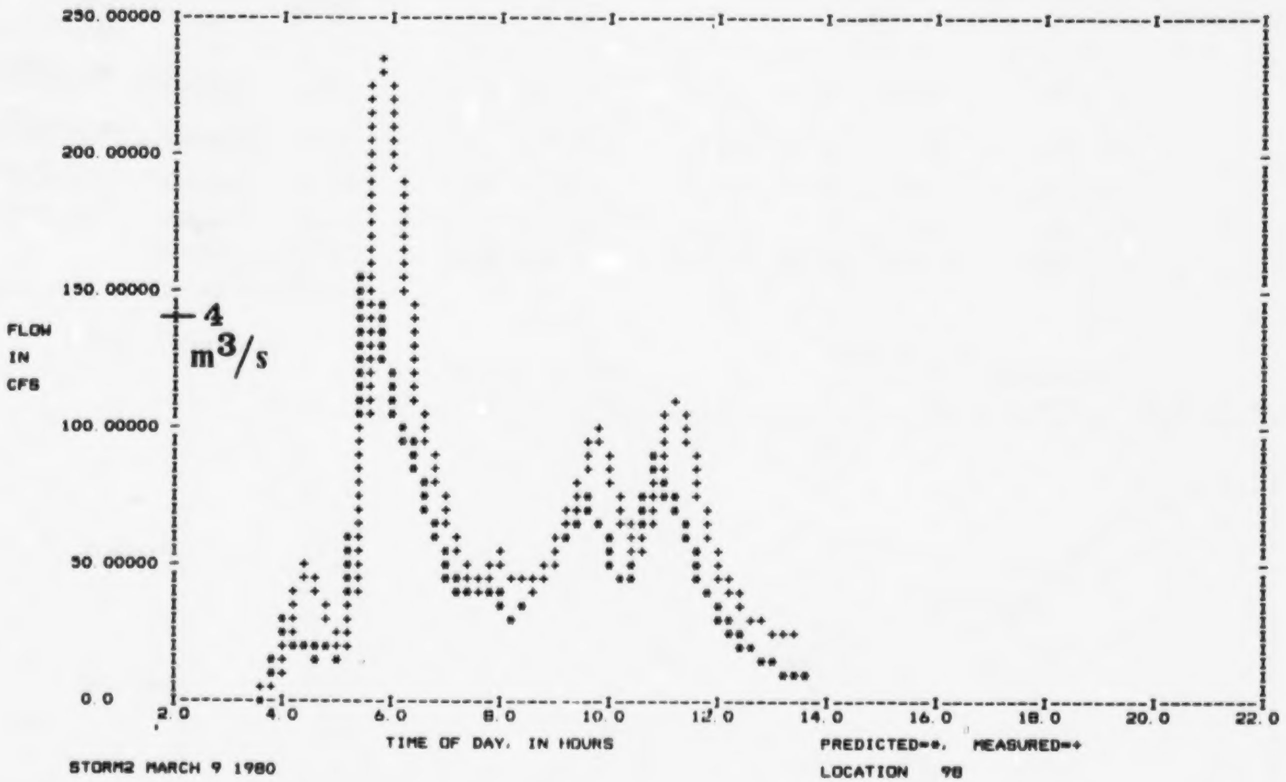

Figure 5. Predicted and measured hydrograph, March 10, 1980 (verification run).



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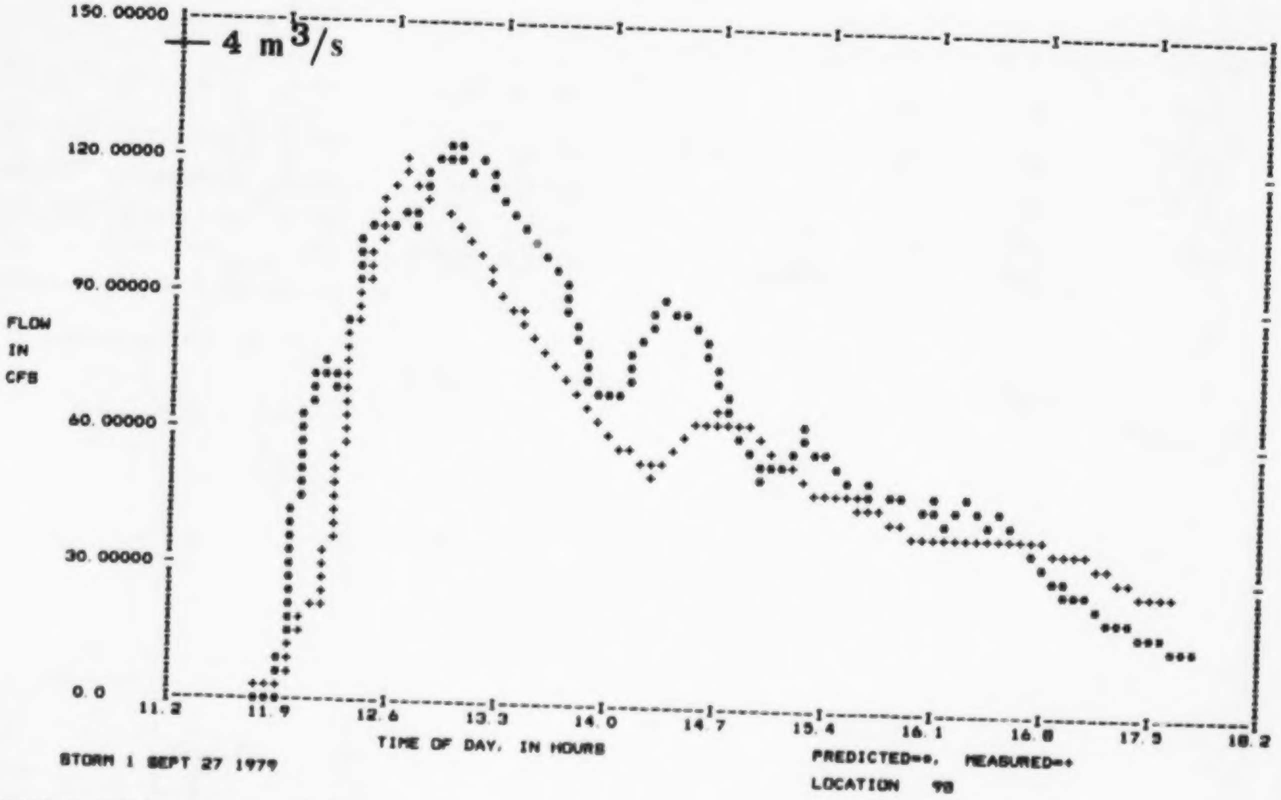
HYDROGRAPH STATISTICS FOR LOCATION  98
                         VOLUME               PEAK FLOW                DURATION              NO.
              CUBIC FEET     INCHES     TIME,HR  FLOW,CFS     START,HR  END,HR  LENGTH,HR    POINTS
PREDICTED,    0.18009E+07     0.222       5.500  158.767        3.500   13.500   10.000       121
TOTAL TIME
MEASURED,     0.23986E+07     0.296       5.750  234.000        3.500   13.417    9.917       120
TOTAL TIME
PREDICTED,    0.17979E+07     0.222       5.500  158.767        3.500   13.417    9.917       120
OVERLAPPING
TIME
MEASURED,     0.23986E+07     0.296       5.750  234.000        3.500   13.417    9.917       120
OVERLAPPING
TIME
DIFFERENCES,
  ABSOLUTE    0.60063E+06     0.074       0.250   75.233
  % OF MEAS                  25.041               32.151
```

Figure 6. Predicted and measured hydrograph, March 9, 1980 (calibration run).



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HYDROGRAPH STATISTICS FOR LOCATION  98

                      VOLUME                 PEAK FLOW                   DURATION
              CUBIC FEET    INCHES      TIME,HR  FLOW,CFS      START,HR  END,HR  LENGTH,HR        NO
PREDICTED,                                                                                      POINTS
TOTAL TIME    0.13682E+07   0.169       13.000   122.838       11.750   17.750    6.000           73
MEASURED,
TOTAL TIME    0.11976E+07   0.148       12.667   119.000       11.750   17.667    5.917           72
PREDICTED,
OVERLAPPING   0.13636E+07   0.168       13.000   122.838       11.750   17.667    5.917           72
TIME
MEASURED,
OVERLAPPING   0.11976E+07   0.148       12.667   119.000       11.750   17.667    5.917           72
TIME
DIFFERENCES,
  ABSOLUTE   -0.16607E+06  -0.021       -0.333    -3.838
  % OF MEAS                -13.868                -3.225
```

Figure 7. Predicted and measured hydrograph, September 27, 1979 (verification run).

and the nine highest runoff volumes (total flows) are indicated in Tables 2 and 3. The hyetographs of 52 and 65 month storms for each case are shown in Figures 8 and 9 and their characteristics are listed in Table 4.

TABLE 2. STORMS RANKED BY PEAK FLOW

Rank	Return Period months	Date	Duration hrs	Depth in	Peak Flow cfs
1	260	9/ 8/68	6	6.52	1907
2	130	7/21/70	34	8.21	1605
3	86.7	7/16/64	73	10.16	1505
4	65	7/21/69	58	6.11	1131
5	52	9/ 3/65	11	4.35	1104
6	43.3	9/20/69	88	13.79	1070
7	37.1	12/ 3/64	61	9.92	1028
8	26	7/ 9/65	81	6.01	807
9	23.6	6/30/64	48	4.67	805

TABLE 3. STORMS RANKED BY RUNOFF VOLUME (TOTAL FLOW)

Rank	Return Period months	Date	Duration hrs	Depth in	Runoff Volume in
1	260	9/20/69	84	13.79	3.69
2	130	3/28/73	108	10.91	2.86
3	86.7	12/ 3/64	61	9.92	2.74
4	65	7/16/64	73	10.16	2.69
5	52	7/26/75	97	9.30	2.39
6	43.3	7/21/70	34	8.21	2.24
7	37.1	8/ 2/66	171	7.72	2.09
8	26	3/31/62	24	7.78	1.96
9	23.6	10/ 6/59	87	7.35	1.95

Tables 2 and 3 show that the return periods of the individual storms are quite different when ranked by another parameter. This is illustrated further for the 5-year storms in Table 5. Storm V-65 is rare by both flow measures; in fact, it is the third largest storm of record on the basis of peak flow (and the second largest rainfall volume event). However, when storm V-52 is ranked by peak flow, and when storm P-52 is ranked by volume, they are seen to be not especially rare, with return periods of 7.4 and 6.8 months, respectively. Return periods for rainfall volumes shown in Table 5 were obtained from the SYNOP program for rainfall frequency analysis (EPA, 1976; Hydrosci-

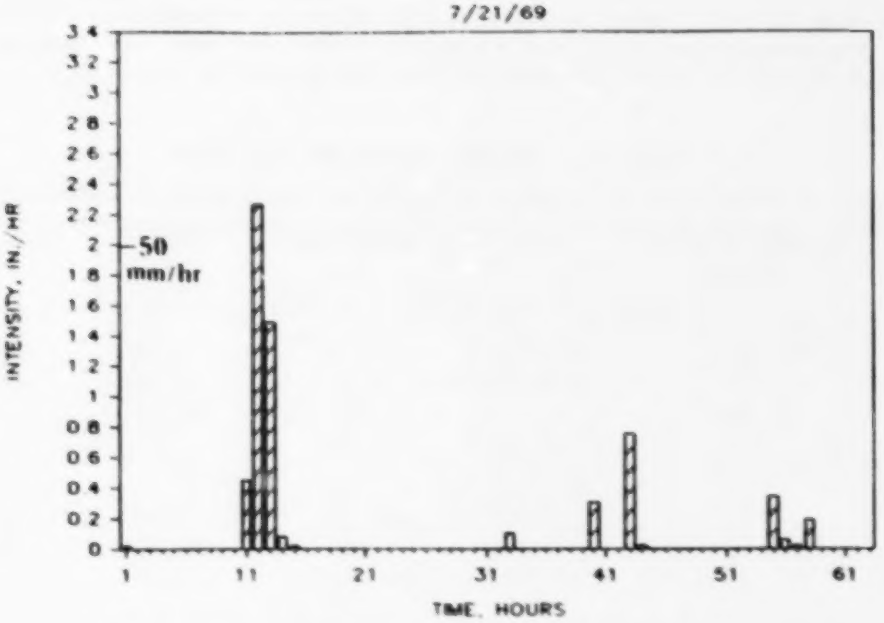

Figure 8a. Hyetograph for storm P-65, July 21, 1969. 65-month return period based on peak flow.

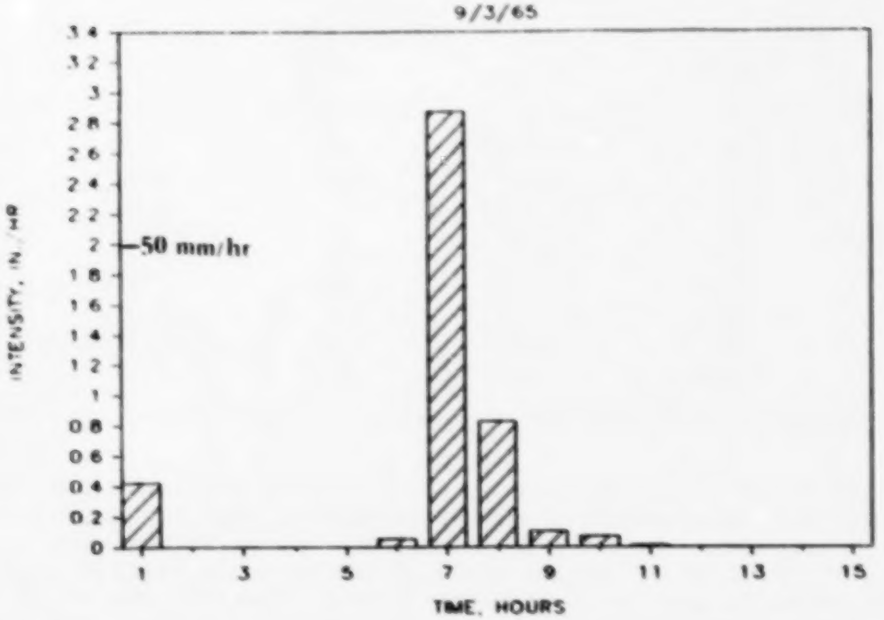

Figure 8b. Hyetograph for storm P-52, September 3, 1965. 52-month return period based on peak flow.

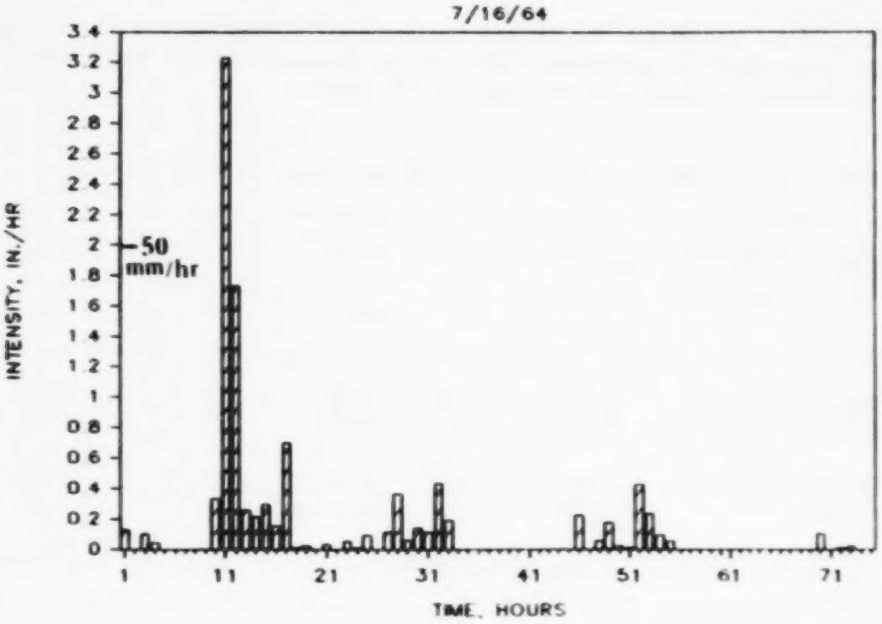

Figure 9a. Hyetograph for storm V-65, July 16, 1964. 65-month return period based on runoff volume.

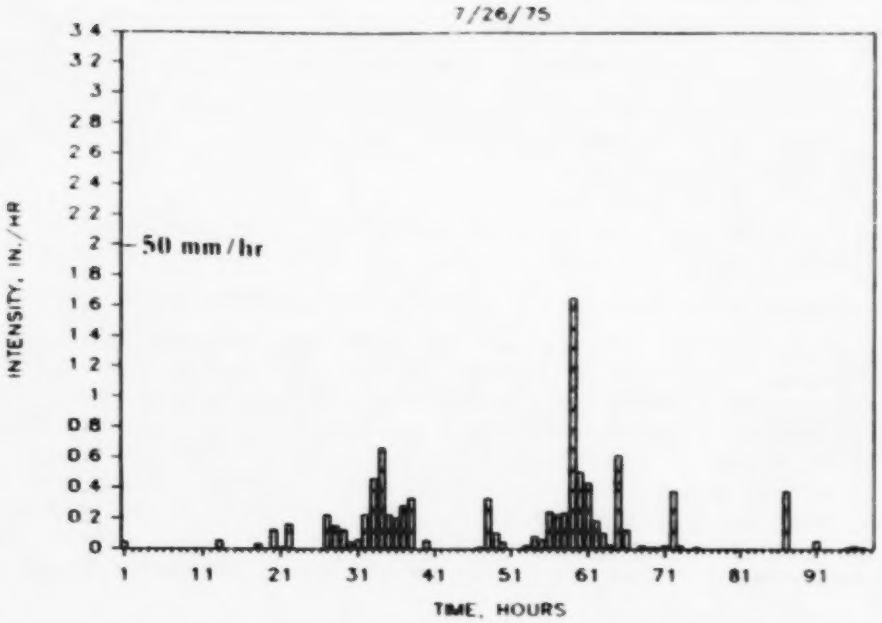

Figure 9b. Hyetograph for storm V-52, July 26, 1975. 52-month return period based on runoff volume.

ence, 1979). Thus, although the return periods were computed slightly differently, Table 5 further illustrates that return periods of the same event ranked by different parameters are rarely the same.

TABLE 4. CHARACTERISTCS OF 5-YEAR HISTORIC DESIGN STORMS

No.*	Date	Runoff Duration hr	Runoff Volume in	Rainfall Volume in	Peak Flow cfs	Time Since Last Event hr	Rainfall Last Event in
V-65	7/16/64	73	2.69	10.16	1670	74	0.27
V-52	7/26/75	97	2.39	9.30	685	31	0.07
P-65	7/21/69	58	1.61	6.11	1131	32	1.35
P-52	9/ 3/65	11	1.19	4.35	1104	67	0.11

* V means ranked by volume, P means ranked by peak, and numbers are return periods in months.

TABLE 5. RETURN PERIODS (MONTHS) OF "5-YEAR" STORMS BY VOLUME, PEAK FLOW AND RAINFALL

No.	Date	Return Period by Volume	Return Period by Peak Flow	Return Period by Rainfall Vol.
V-65	7/16/64	65	87	156
V-52	7/26/75	52	7.4	78
P-65	7/21/69	14	65	12
P-52	9/ 3/65	6.8	52	7.8

Seven of the historic storms defined above were then run again through the model (storms for return periods of 37.1 and 43.3 months were omitted) using hourly rainfall inputs but a 5-min time step, instead of the hourly time step used in the continuous simulation. In lieu of continuous simulation for these detailed analyses, 4 to 6 days of prior rainfall were run through the model prior to the beginning of each design event. This "pseudo-continuous simulation" thus accounts for antecedent conditions without the need for running the entire rainfall time series at a short time step. Results will be described following a discussion of the generation of synthetic design storms.

SYNTHETIC DESIGN STORMS

Following the techniques of Arnell (1982), five synthetic design storms were constructed for comparision with the historic storms in Tallahassee: Soil Conservation Service (SCS, 1964), Chicago (Keifer and Chu, 1957), Illinois State Water Survey (Huff, 1967), Sifalda (1973), developed in Czechoslovakia, and Flood Studies Report, FSR, (Natural Environment Research Council, 1975), developed in Great Britain. The choice of a 24-hour duration for the storms was made for two reasons: 1) it is commonly used in engineering practice in the Tallahassee area, and 2) approximate calculations of the time of concentration of the basin using the kinematic wave equation (Eagleson, 1970) yielded estimates ranging from 13 to 60 hours, depending on the choice of rainfall excess. Thus, 24 hours is at least in the range of possible times of concentration. But the very idea of having to arbitrarily select a storm duration in the first place illustrates one of the major difficulties in using synthetic design storms.

Five-year storms are considered as an example. From the Tallahassee region IDF curves (Weldon, 1985), the 5-year, 24-hour average intensity is 0.305 in/hr, giving a 5-year, 24-hour depth of 7.32 in. The five synthetic design storms were then scaled to produce this total storm depth. Various 24-hour depths from the IDF curves are compared with historic storms in Table 6. These were compiled from the SYNOP program (EPA, 1976) with a minimum interevent time of 5 hours and thus differ in magnitude slightly from the rainfall volumes identified from the frequency analysis of runoff conducted using SWMM. It may be seen that although the IDF and historic depths are comparable, the actual durations of the historic storms are certainly not 24 hours.

TABLE 6. COMPARISON OF 24-HOUR IDF DEPTHS WITH SYNOP FREQUENCY ANALYSIS OF HISTORICAL RAINFALL EVENTS

Storm Date	Return Period yr	Duration hr	Depth in
9/20/69	26.0	54	13.41
IDF	25	24	10.08
7/17/64	13.0	33	9.76
IDF	10	24	8.64
12/13/64	8.7	36	9.73
7/28/75	6.5	53	8.84
7/21/70	5.2	20	8.18
IDF	5	24	7.44
8/30/50	4.3	36	7.34
6/18/72	3.7	46	7.17

The final group of synthetic hyetographs is shown in Figures 10 and 11. It is clear that they bear no resemblance to the actual historic storms shown in Figures 8 and 9.

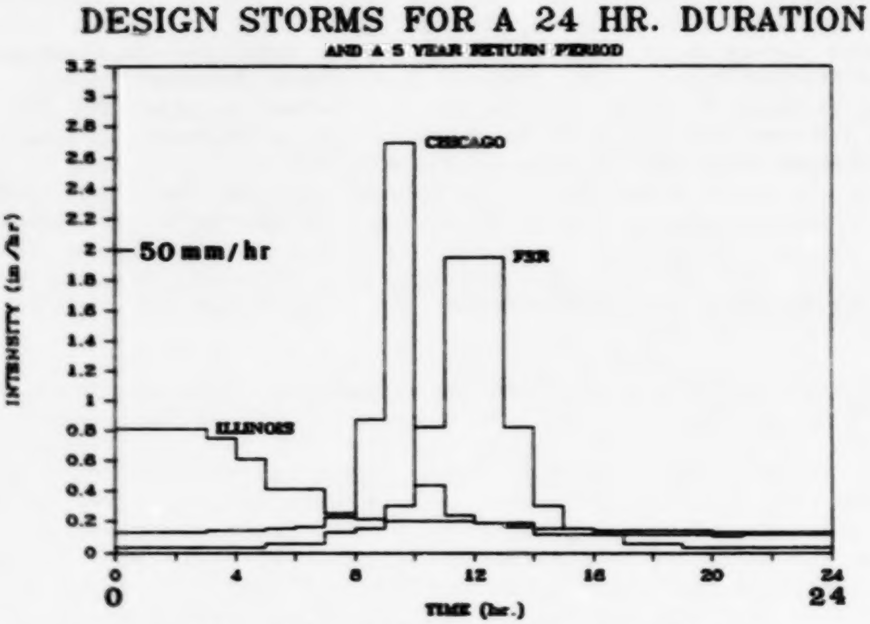

Figure 10. Chicago, Flood Studies Report, and Illinois design storms.

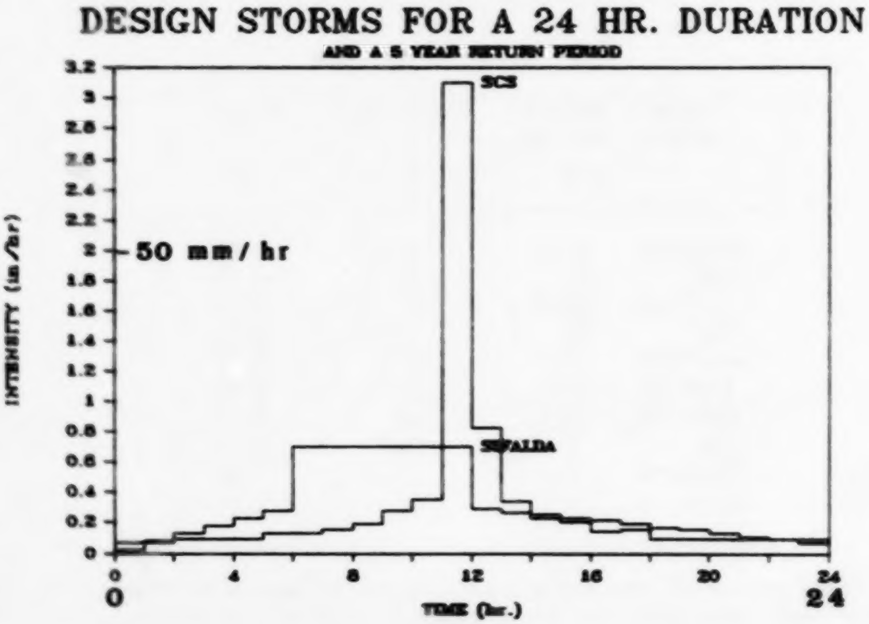

Figure 11. Soil Conservation Service and Sifalda design storms.

OTHER DESIGN TECHNIQUES

The USGS has performed a flood frequency analysis on 15 basins in the Tallahassee area using the combination of continuous simulation and regression discussed earlier (Franklin and Losey, 1984). Their predicted 5-yr peak for Megginnis Arm is 1570 cfs.

Finally, the venerable Rational Method could be applied. The results for this method are also shown below, using a runoff coefficient of 0.27 (from the SWMM modeling) and Weldon's (1985) IDF data.

RESULTS

The main objective is a comparison of peak flows developed using alternative techniques. For this purpose, seven of the historic storms listed in Table 2 (with antecedent conditions included) and five synthetic storms were run on a single event basis using the calibrated version of SWMM described earlier. Seven historic storms from Table 3 (for total flow) were also simulated. (Storms for return periods of 43.3 and 37.1 months were not run.) For all runs, a time step of 5 minutes was used, but with hourly hyetograph values, since historic rainfall data at finer time increments were not available in time for this analysis. (Copies of the microfilmed weighing-bucket charts must be ordered from the NWS National Climatic Data Center in Asheville, NC in order to obtain, say, 15 minute hyetograph increments.) Peak flows calculated by the model in this way are somewhat higher than for the identical storms during the continuous simulation because SWMM calculates average flows over the hourly time step during continuous simulation (in order to avoid large continuity errors), whereas instantaneous flows at the end of a time step are calculated during single event simulation. Results for the 5-year storms are given in Table 7 along with results for the other methods that have been applied to this basin. Results for all storms are shown in Figure 12. (The peculiar shape of the curve for runoff volumes is because the volumes are plotted at return periods corresponding to a volume ranking, not a peak flow ranking.)

Before discussing the results, it is interesting to note that when run on a single event basis, the P-65 storm produces a lower peak than does the P-52 storm, in contrast to the results for the same storms during the continuous simulation (Tables 2 and 4). This is an artifact of the numerical methods used in the SWMM Runoff Block. For continuous simulation, average flows over the time step (usually one hour) are computed in order to avoid continuity errors when using long time steps. For single event simulation, instantaneous flows at the end of the time step are computed. The peak instantaneous values are higher than the averages and respond more directly to peak rainfall intensities. This factor, coupled with the shorter time step used for the detailed simulations were enough to alter the rankings of these two storms. In general, all the peak flows predicted during the single event simulations of the historic storms were higher by as much as a few hundred cfs than those predicted during the continuous simulation, but the only change in rankings is the one discussed above.

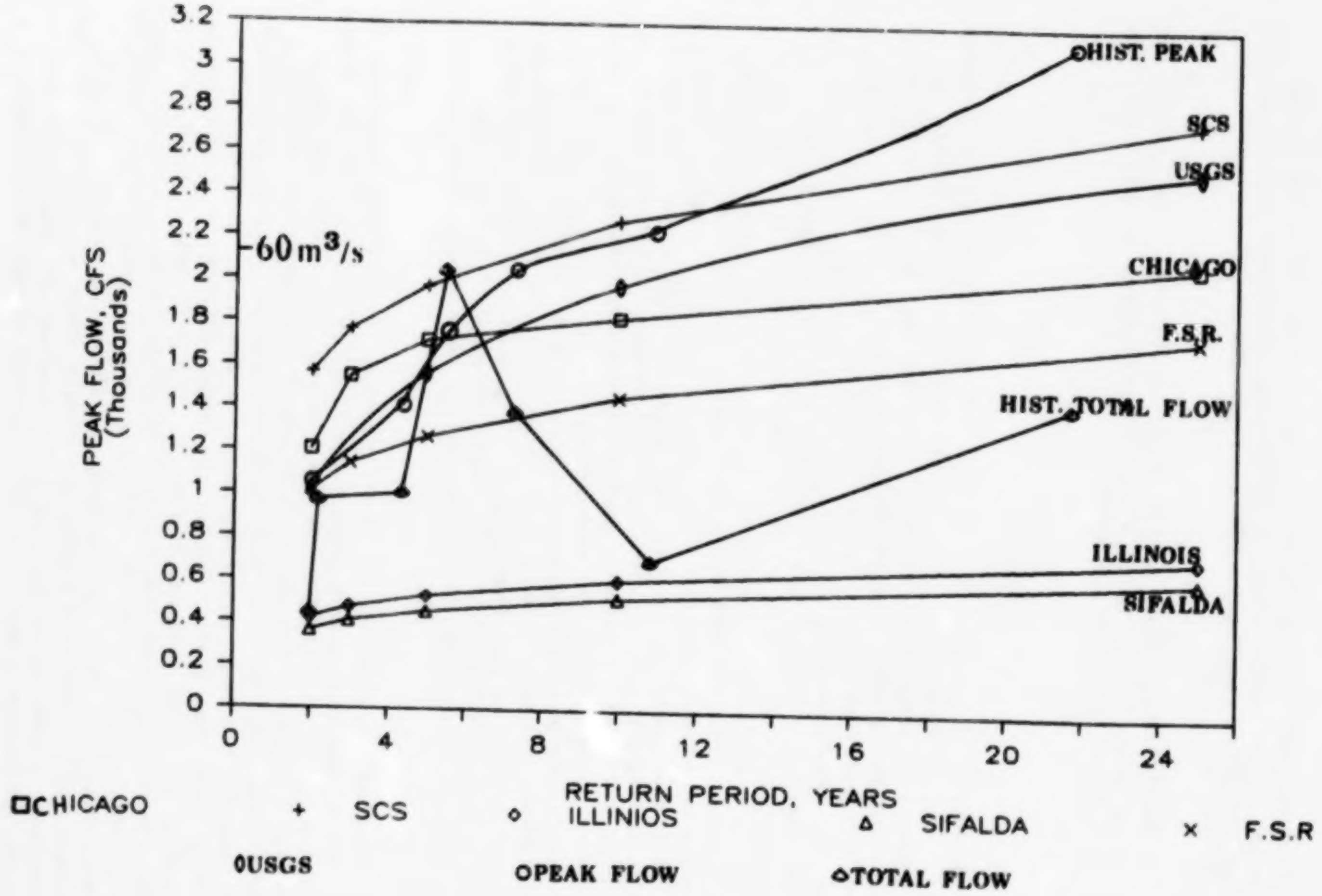

Figure 12. Peak flows versus return period predicted using historical storms, synthetic design storms and the USGS method. Historical storms based on total flow are plotted at return periods corresponding to runoff volume, not peak flow.

Considering a 5-year return period, for the Rational Method to predict a peak flow of 1748 cfs would require a duration on the IDF curve of approximately 50 min, much less than the actual (but unknown!) time of concentration. Hence, it does not appear to be applicable. Furthermore, the return period for the method is assigned on the basis of conditional frequency analysis of rainfall, as oppposed to the storm event analysis of runoff peaks or volumes of the other methods.

TABLE 7. COMPARISON OF 5-YEAR PEAK FLOWS USING SEVERAL METHODS

Storm	Peak Flow cfs	Method	Peak Flow cfs
V-65	1996	USGS	1570
V-52	983		
P-65	1384		
P-52	1748	Rational Method	
		t_c = 24 hours	187
SCS	1926	t_c = 13 hours	289
Chicago	1677	t_c = 50 min	1764
Illinois	511		
Sifalda	441		
F.S.R.	1235		

Somewhat unexpectedly, the results for the synthetic design storms fall on the low end of the estimates; only the SCS storm is consistently high. For example, a similar study by Marsalek (1978) showed higher peaks using the synthetic storms, but his basins were much smaller (64 acre and 321 acres). The difference may be partly due to the choice of a hyetograph interval of 1 hour; synthetic hyetographs are sensitive to this choice since as the interval becomes smaller, the peaks for shapes such as the Chicago and SCS storms tend to become very high (and thus produce higher runoff peaks). This is another complication in the use of such methods. On the other hand, the historic rainfall was also discretized at 1-hour intervals and might also be expected to have higher peaks at shorter intervals (but the shorter interval data were not available for this study). Furthermore, the Megginnis Arm basin is large with a long time of concentration; in principle, short time increment fluctuations should be well damped by the basin response.

Another reason why the peaks are higher with the historic storms lies in the differences in storm volumes and durations. For instance, the two "5-year" historic storms chosen on the basis of volume are both larger than the 24-hr IDF value of 7.32 inches. On the other hand, historic storm P-52 has a duration of only 11 hours in which to concentrate the rainfall of 3.93 inches. The anomaly is historic storm P-65, which has a lower rainfall volume and a longer duration than do the synthetic storms — but it has a pocket of high intensity rainfall (Figure 8a) coupled with wet antecedent conditions.

The main difference between the results of this study and those of Marsalek (1978) is in the manner of choosing the historic storms. Marsalek isolated storms on the basis of rainfall criteria only, including a requirement that storms have a total depth greater than 4.9 in/hr and/or contain a 10-minute intensity greater than 5.9 in/hr. His historic storms were further screened by seeking maximum depths for durations of 10, 15, 30 and 60 minutes. In this study, the storms were screened solely on the basis of the continuous simulation, allowing the catchment to filter out the important elements of the rainfall-runoff response (to the extent that it is properly modeled in SWMM).

Arnell (1982) found that the Sifalda storm gave the best agreement with historic storms, in contrast to this example in which it gives a low estimate for peaks. In general, this catchment produces higher peaks for storms with late and high peak rainfall intensities, appearing after the infiltration capacity of the basin is satisfied.

The USGS estimates (based on a log-Pearson Type 3 frequency analysis of simulated flows) do not differ widely from those of the historic SWMM runs at low return periods, but all methods give lower peak flows at high return periods than that predicted by SWMM for the 260-month peak-flow storm. As can be seen in Figure 12, the SWMM simulations of the historic storms produce lower peaks than those of the USGS and the Chicago and SCS design storms at low return periods. Hence, it cannot be said unequivocally that synthetic design storms produce conservative results. On the basis of this single study on a single catchment, the SCS and Chicago storms are conservative at low return periods but not at high ones.

However, there is considerable uncertainty as to the true return period of the largest storm runoff peak in the 22 year period. Assuming on the basis of order statistics that the empirical frequencies assigned to the historic storms have a beta distribution (Gumbel, 1958, Chapter 2), then +/- 25% confidence intervals about the expected value of the return period of the largest peak (23 years) are 11.1 and 47.5 years. That is, the probability is 50% that the true return period of the largest simulated peak in the 22 year record lies between 11.1 and 47.5 years. These confidence intervals narrow considerably at lower return periods. Hence, it can be said with more confidence that the SCS and Chicago storms are conservative at low return periods than that they are not conservative at high return periods. A much longer rainfall record would resolve the question at the 22 year return period, but there would always be doubt at the end point of the record as to the proper frequency (even if a frequency distribution, such as the log-Pearson Type III, were fit to the data). A main result of this study is simply that there is no guarantee that synthetic storms are conservative over the whole range of return periods.

Ideally, the measured runoff records should be analyzed to isolate storm events and resolve the question of which of the several curves shown on Figure 12 is correct. But the gage for the Megginnis Arm Catchment has only been in operation since 1973, making it difficult to identify storms for return periods greater than about 10 years. Another difficulty is the changing land use within the catchment, with rapid urbanization altering the nature of the rainfall-runoff response. This factor tends to direct the analysis back to a properly calibrated model. However, for the record, the largest peak observed during the intermittent sampling since 1973 is 724 cfs on May 23, 1980 from a storm of 1.43 inches. It is not suprising that the much higher rainfalls used in the simulations would produce peaks on the order of 2000 cfs (Figure 12). In general, maximum rainfalls after 1973 have been much less than before (see Table 6). Thus, peak flows simulated using 1958-79 rainfall data are quite likely to be higher than those measured since 1973.

HISTORIC VERSUS SYNTHETIC STORMS

The reader will probably not be surprised to learn that the authors favor the historic storm values for the following reasons:

1. They are based on a frequency analysis of peak flows (or runoff volumes), not on a conditional frequency analysis of rainfall depths. That is, the frequency analysis is on the parameter of interest.

2. The frequency analysis of the continuous time series of flows includes all effects of antecedent conditions whereas the synthetic design storms do not. Moreover, the detailed single-event simulations of historic storms may be conducted in a "pseudo-continuous manner," that is, by simulation of several days of antecedent weather prior to the beginnning of the storm of interest.

3. The historic storms avoid the vexing questions of storm duration, shape and hyetograph discretization. The duration is especially critical, since peak flows may arise out of a storm that lasts several days (e.g., storms V-65, V-52, P-65) or out of a short, intense one (e.g., storm P-52). There is simply no basis for establishing a standard duration, such as 24 hours, for all design work, as seems to be the unfortunate tendency in Florida.

4. Historic storms can also be used for analysis of volumes for design of basins for detention or retention. The volume of synthetic storms is arbitrarily linked to the assumed rainfall duration. A given volume can result from an infinite number of combinations of intensity and duration, with a corresponding range of return periods.

5. If historic storms are used for design, the local citizenry can be confident that the design will withstand a real storm that may be remembered for its flooding by many, as opposed to an "unreal" synthetic storm. Thus, the engineers or agency may make a statement such as, "our design will avoid the flooding that resulted from the storm of September 3, 1965."

PROBLEMS WITH USE OF HISTORIC STORMS FOR DESIGN

The analysis outlined in this paper is neither brief nor simple. A continuous simulation model must be calibrated and used, first to identify storm events and second to perform the more detailed design simulations. This means more work both for the design engineers and for the reviewing agencies (leading to the common arguments in favor of synthetic design storms -- they are quick, easy, and consistent from application to application). A possible compromise would be to compile an atlas of historic storms and their antecedent conditions for use in design, based on generic runs of a continuous model for representative catchment sizes, land uses, soil types, climatic regions etc. Research on this possibility is underway at the University of Florida.

Another consideration is the high spatial variability of real storms and the sensitivity of runoff to storm direction (James and Shtifter, 1981; Surkan, 1974). Historic storms must still be derived from point rainfall data. The catchment may be more sensitive to the spatial dynamics of storms than to the relative rankings within a time series. Of course, the same remarks apply to synthetic storms. However, the analysis of historic storms may be carried further and dynamic storms constructed from historical data (James and Scheckenberger, 1984). Spatial dynamics are more important as the size of the catchment increases.

Finally, the question of economics versus safety factors raised early in this paper is unanswered. Is there a savings in construction costs by using the type of analysis outlined in this paper (with the implication that the synthetic design storms commonly used in design are too conservative)? Or are the synthetic design storms not conservative and therefore providing less than the level of protection assumed by design engineers? What are the implications on flood stage as opposed to peak flows? Stage may only be affected in a minor way by differences of several hundred cfs in flow for a wide flood plain. In such cases the design may be insensitive to the design methodology used, and the quickest and fastest method may suffice. These questions are also under investigation at UF.

CONCLUSIONS

Given a calibrated and verified model, selection of historic design storm events from a continuous simulation offers the advantages of a meaningful frequency analysis on the runoff parameter of interest, such as peak flow or runoff volume; the frequency analysis need not be tied to rainfall frequencies as with the use of synthetic design storms. For this study, historic design storms were selected from a frequency analysis of peak flows and runoff volumes on the basis of a continuous SWMM simulation using the 22 year period of available rainfall. The model had previously been calibrated and verified using local rainfall-runoff data for the Megginnis Arm Catchment in Tallahassee. The historic storms were then simulated and the results compared to those obtained using five different synthetic design storms, including the widely used SCS Type II storm. In general, the SCS and Chicago storms produced higher peaks at low return periods than produced by the historic storms, while the historic storms produced a higher peak at the highest return period. Thus, it cannot be assumed that the synthetic storms are conservative over the entire range of return periods.

ACKNOWLEDGEMENTS

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An Expert System Prototype for RECEIV-II Using TURBO PASCAL

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Abstract

A pre-processor written in TURBO PASCAL was created for the RECEIV-II model. The graphics based pre-processor enables trained engineers and scientists familiar with estaurine hydrodynamics to easily run the receiving water model. The pre-processor acts similarly to programs such as LOTUS-123 in enabling a higher percentage of people to become competent modelers.

Introduction

The use of the personal computer by the engineer and scientist is a landmark advance that greatly increases their analytical tools. Spreadsheets, Word Processing programs, and Data Base Management programs increase productivity by (1) compressing the time between the initial conceptualization and the final product; and (2) eliminating some of the mechanical impediments to original creation. The PC aids both in the creation and production of engineering and scientific products.

The primary usage of the PC, however, is restricted to programs equally applicable to any profession, trade, or occupation. The scientific and engineering community has only begun to realize the potential of the PC as design aids and expert systems.

The purpose of this paper is to introduce a prototype expert system for the RECEIV-II model (Raytheon, 1974) constructed using the Borland International program TURBO PASCAL (Borland, 1985).

Expert Systems

An expert system incorporates the human expert's knowledge into a computer program so that others can solve the same type of problem (Van Horn, 1986). Some of the characteristics of "real" expert systems as described by Van Horn(1986) are: (1) the program contains the heuristic knowledge of an expert; (2) the program is able to interface with the user and developers in natural language; (3) the program asks questions to obtain needed data; (4) the program is easily refined and upgraded without extensive reprogramming; (5) the program can explain its conclusions; (6) the program can accept uncertain input and assign it a certainty factor; (7) the program gives answers with a certain level of confidence; and (8) the program learns from its own performance.

A combination pre-processor and water quality model on the PC addresses most of the requirements of an expert system using a liberal interpretation of the definition of an expert system. The model itself functions as the rule base of the expert system. The inference engine is built into the structure of the program, and the pre-processor acts as a quasi natural language interface.

The key gain in using a pre-processor and model in the PC environment is the increase in modeling competence, performance, and productivity. The model can now be used by any engineer or scientist with a knowledge of coastal hydrodynamics and limited computer experience.

Pre-Processor

This section will explain the actual construction of a menu driven pre-processor in TURBO PASCAL. The tools used to construct the pre-processor include TURBO SCREEN (Pascom Computing, 1985), TURBO PASCAL (Borland International, 1985), and TURBO Toolbox (Borland International, 1984).

TURBO SCREEN is a program used to construct graphical menus for data input. The menus are made using a menu driven text editor with WORDSTAR commands. The program generates TURBO PASCAL code that will mimic in a compiled TURBO PASCAL program the menus created by the programmer. The menus, which consist of colored or shaded foreground text on colored or shaded backgrounds, are restricted to 24 row by 80 columns. The text explains the data input requirements for each data field and function as built in help screens. The pre-processor data fields, programmer defined, are either real, integer, string or character.

A graphical menu approach to the pre-processor was chosen over a simple question and answer approach for aesthetics, program modularity, and the ability to use window management. Each menu is defined as a picture or window and is manipulated as a record in PASCAL. PASCAL records are similar to arrays in FORTRAN except records can have mixed types, e.g. integer, real, and string components in the same record. Each menu is easily manipulated by the programmer by simply specifying the picture number. The beauty of PASCAL over BASIC and FORTRAN is this ability to use records.

The pre-processor either creates a new input file or edits an existing file. The user of the RECEIV-II model does not have to learn the FORTRAN field codes for data input; the pre-processor handles the actual ASCII file manipulation. The pre-processor allows movement between fields in the screen by reading the keyboard buffer. The data input stream does not have to be sequential. It can be entered vertically or horizontally in the menu. As an example entering channel data can be done channel by channel or field parameter by field parameter.

The creation of the pre-processor (1950 PASCAL statements) took approximately three days from start to working product using the advanced programming tools available from Borland International. The program creation was greatly aided by the sequential nature of the menus built using TURBO SCREEN and the editing environment of TURBO PASCAL.

Discussion

Compilers constructed specifically for the PC such as TURBO PASCAL and TURBO PROLOG (Borland, 1986) offer an enhanced editing environment, faster compilation, and the advantages of a higher level language. The programmer's productivity is increased because: (1) low level language errors such as undefined variables are eliminated; (2) there is better error checking during compilation; and (3) faster compilation speeds (less than 1 minute for a 2000 line program). The structure of PASCAL is similar to FORTRAN. A FORTRAN programmer can easily learn the ancillary techniques to program in PASCAL.

The most expensive component of software development is the programmer's time in writing and debugging source code (Brooks, 1975). Languages and compilation systems that enhance the creation and implementation of programs will: (1) increase programming productivity; and (2) enable a higher percentage of the population to program. As experienced FORTRAN programmer's we know the frustration of late night sessions hunting for a bug in the program. A language that eliminates most bugs in the compilation stage has a tremendous evolutionary advantage over lower level languages.

Over the last 40 years as computer languages have advanced from machine code to assembly language to BASIC and FORTRAN to PASCAL to MODULA-2 the ease of programming has increased and the percentage of the population capable of programming has increased. The popularity of programs such as LOTUS-123 and DBASE-III is partly due to the need of everyone to appear as competent programmers and computer "literate". These programs allows a novice programmer the almost instantaneous ability to generate programs.

A pre-processor linked to a model on the PC allows not just the experienced programmer but the trained engineer and scientist to be a competent modeler. The "natural language" interface is similar to LOTUS-123 in permitting a higher percentage of the population of engineers and scientists to become competent RECEIV modelers.

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MODELING FLOOD HYDROLOGY USING HYMO

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ABSTRACT

HYMO is a computer program well suited to model flood hydrology for a wide range of watershed conditions. Any design storm duration and distribution can be input along with site-specific unit hydrograph shape parameters. Stream or channel routing is accomplished using the variable storage coefficient method; the storage-indication method is used for reservoir routing. The original program was developed for mainframe equipment, but can easily be adapted for use on microcomputers.

After a brief summary of HYMO commands, formatting, and storage, the paper describes hydrograph analyses performed in the U.S. Virgin Islands. Results of the analyses were used to develop a basis for assigning unit hydrograph parameters to model flood hydrology.

HYMO DESCRIPTION

HYMO, derived from the words hydrologic model, is a computer language developed by the Agricultural Research Service, U.S. Department of Agriculture, in cooperation with the Texas Agricultural Experiment Station, Texas A&M University (1). The original program was developed for mainframe equipment, but can easily be adapted for use on microcomputers. HYMO commands are simple to use and offer a great deal of flexibility. For example, any design storm duration and distribution can be input along with site-specific unit hydrograph shape parameters. Flood hydrographs are developed using unit hydrograph theory and the SCS rainfall-runoff relationship.

The variable storage coefficient (VSC) flood-routing method is used for stream routing. The VSC method accounts for changes in channel velocity or reach travel time with flood stage. Although an iterative solution is used, the VSC method requires minimal computer time and is free of convergence problems. Channel section rating curves used for VSC flood-routing calculations can be input directly or computed internally by HYMO for uniform flow using Manning's Equation. Channel section data for internal calculations can be segmented into flood plain and main channel components with appropriate slope and n values for each segment.

The storage-indication method as documented in the SCS National Engineering Handbook, Section 4, Hydrology (2) is used by HYMO for reservoir routing computations.

HYMO COMMANDS

The HYMO language includes 17 commands for performing flood hydrology and sediment yield calculations. Commands are expressed in the first 20 columns of the data card, with columns 21 through 79 used for numeric data and keywords. Column 80 is reserved for a page change code (an asterisk in column 80 causes the printer to advance to a new page). Continuation cards are allowed when 59 characters are insufficient to present the data.

The data can be written in any format, but at least one blank space must be left between data items. A decimal is required for numbers containing fractions, but not for whole numbers. Keywords can be written with the data to describe individual data items. Comment cards may be used at any point in a HYMO program by punching an asterisk in column 1 and the comment in columns 2 through 79.

Six hydrographs can be stored in a HYMO program at a time, identified by storage location numbers 1 through 6. The storage location numbers must be repeated for watersheds requiring more than 6 hydrographs. However, no more than six hydrographs are ever needed at one time because HYMO programs begin at the head of a watershed and work downstream through one reach at a time. Because the first hydrograph is lost when a storage location number is used to store or compute another hydrograph, the user should be sure that the hydrograph will not be referred to again before using the storage location number for another command.

Details of rules for HYMO commands are presented in the Users Manual (3) along with an example problem.

HYDROGRAPH ANALYSIS

To establish a site-specific basis for using HYMO in the U.S. Virgin Islands, published and unpublished streamflow data were obtained from the U.S. Geological Survey (4)(5). Because long unbroken periods of record were not available, observed data did not provide an adequate basis to develop an annual maximum flood series. The data did, however, contain several isolated single-event runoff hydrographs which could be analyzed to determine site-specific flood hydrograph parameters. These included the time to peak, the total runoff volume, the peak flow rate, and the SCS unit hydrograph shape factor.

Streamflow data from four gaged watersheds on St. Croix were used to establish a basis for assigning unit hydrograph parameters for ungaged watershed areas. Streamflow records considered in this study are summarized in Table 1.

TABLE 1. USGS STREAMFLOW DATA AVAILABLE FOR ST. CROIX, U.S. VIRGIN ISLANDS

Station Name	Station Number	Drainage Area (mi^2)	Period of Record Examined
River Gut at River	3320	1.42	1963 - 1967 (5 years)
River Gut at Golden Grove	3330	5.12	1963 - 1971 (9 years)
Jolly Hill Gut at Jolly Hill	3450	2.10	1963 - 1968 (6 years)
Creque Gut above Mount Washington Reservoir	3470	0.50	1965 - 1967 (3 years)

The first step in the hydrograph analysis was to obtain the original stage hydrographs (strip charts) from the USGS files in Puerto Rico (6). Strip charts were obtained for selected days for all four stream gaging stations listed in Table 1. These strip charts were then screened for single-peak events which resulted from short duration rainstorms. Since synchronized rainfall records were not available at these gages, this selection process was largely a matter of judgment.

Once the stage hydrographs were selected for analysis, they were converted to discharge hydrographs by application of the appropriate rating table, and the following hydrograph parameters were measured:

1. Time to peak (T_p), in hours, defined as the time from the beginning of rise to the peak of the hydrograph
2. Peak flow rate (q_p) of the runoff hydrograph, in cfs
3. Total volume of runoff (Q), in inches

Using these measured parameters, the hydrograph shape factor, B, was computed for each event as follows:

$$B = \frac{q_p T_p}{AQ} \tag{1}$$

where:

B = Hydrograph shape factor
A = Watershed drainage area, in square miles
(all other terms are as previously defined)

These parameters are reported in Table 2 for each event analyzed.

TABLE 2. HYDROGRAPH ANALYSIS RESULTS, BASED ON USGS DATA

Location	Event Date	T_p (hours)	Q (inches)	q_p (cfs)	B
River Gut at River, No. 3320	4/06/65	1.7	0.033	11.44	415
	5/30/65	3.0	0.064	11.78	389
	11/09/65	1.0	0.023	18.91	579
Mean		1.9			461
River Gut at Golden Grove, No. 3330	8/01/63	4.00	0.240	144.0	467
	11/02/63	1.30	0.100	123.0	311
	11/21/63	1.25	0.021	28.1	325
	10/24/69	1.50	0.112	169.0	440
	10/30/69	5.50	0.062	26.5	457
	12/02/69	2.75	0.070	41.5	317
Mean		2.72			386
Jolly Hill at Jolly Hill, No. 3450	1/03/63	0.60	0.024	39.5	470
	1/04/63	0.50	0.097	177.0	434
	12/12/65	0.60	0.009	17.5	556
	12/12/65	0.70	0.023	39.5	572
Mean		0.60			508
Creque Gut above Mt. Washington Reservoir, No. 3470	10/13/65	1.0	0.297	91.1	613
	11/12/65	1.0	0.250	79.8	638
	11/09/65	0.5	0.174	83.7	481
	5/17/65	0.5			462
Mean		0.75			549

Equation 1 is an algebraic transformation of the hydrograph peak rate equation employed by the Soil Conservation Service (SCS) (7). The standard value for B used by the SCS in the majority of hydrologic design applications is 484. However, Mockus (1972) reported that the hydrograph shape factor has been known to vary from about 600 in steep terrain to 300 in very flat swampy country. From equation 1, it can be seen that the larger the hydrograph shape factor, B, the larger the peak rate of runoff generated by a given volume of runoff, Q. Thus, the hydrograph shape, which is generally related to topography, is also an important factor influencing flood potential.

The hydrograph shape factor is a watershed characteristic and should be a constant for each watershed analyzed. However, considerable variation is reported in Table 2 for individual event B values at each gaging station. These variations are primarily due to the fact that all errors present in each measured hydrograph parameter (i.e., T_p, Q, and q_p) are combined in the computation of B. The time to peak, T_p, is a particularly difficult parameter to measure accurately due to the time scale of the original strip charts (1 inch = 10 hours) and the short times of observation. Therefore, the mean of the event B values is probably the best estimate of the hydrograph shape factor for each watershed analyzed.

Plotting the results from Table 2 as shown in Figure 1, the relationship between the unit hydrograph shape factor, B, and drainage area provides a basis for modeling ungaged watersheds. A total of 13 watershed areas in the U.S. Virgin Islands ranging in size from 365 acres to 3,396 acres have been modeled using this procedure. Nine of these watersheds were on St. Croix and four on St. Thomas. The results of this modeling work are contained in three technical reports by CH2M HILL (8)(9)(10).

CONCLUSIONS

HYMO is a practical tool for evaluating flood hydrology for a wide range of conditions and project requirements. The analysis of observed streamflow hydrographs, as demonstrated using data for the U.S. Virgin Islands, can provide a site-specific basis for assigning unit hydrograph parameters for ungaged watersheds.

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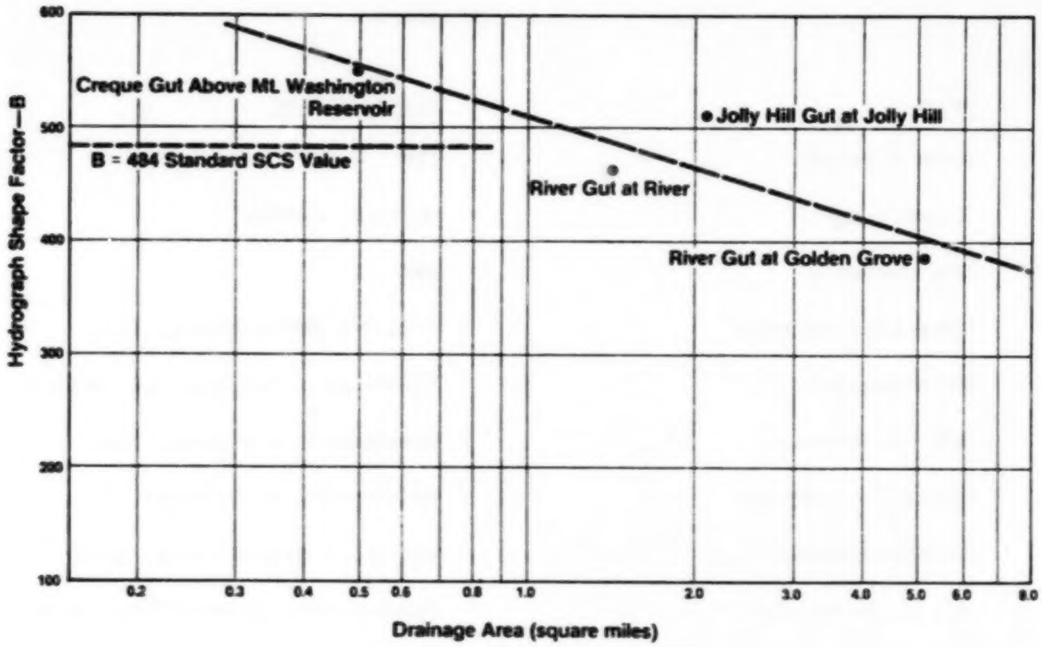

FIGURE 1. Hydrograph Analysis Results for the U.S. Virgin Islands.

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